Cost Implications for Including Tsunami Design in Mid-Rise Buildings along the US Pacific Coast

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Abstract

Tsunami design requirements for construction in the United States of America have been added to the current edition of ASCE/SEI 7-16, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures". This standard, including the new Chapter 6 on Tsunami Loads and Effects, was adopted by the International Building Code, IBC-2018, which will soon apply to most communities throughout the US. Tsunami design is required for all Tsunami Risk Category (TRC) III and IV buildings located in the mapped Tsunami Design Zone (TDZ) along the coastlines of the five US Western States, namely Alaska, California, Hawaii, Oregon and Washington. Local jurisdictions are encouraged to require tsunami design for TRC II buildings within the Tsunami Design Zone if they have sufficient height to provide "refuge-of-last-resort" for those who cannot evacuate to high ground. The improved performance of these structures during future tsunamis will also increase the resilience of these communities. This study evaluates the cost premium if tsunami design is required for mid-rise commercial and residential reinforced concrete buildings located in the TDZ.

Contents

Introduction

The Indian Ocean Tsunami in December 2004 caused extensive damage to communities around the Indian Ocean and resulted in the loss of over 240,000 lives. This event triggered a significant shift in tsunami preparation, warning systems and research. Prior to 2004, most tsunami research was focused on open ocean propagation of tsunami waves in order to predict better the arrival time and the potential size of the tsunami wave along affected shorelines. The focus was primarily on saving lives in coastal communities through horizontal evacuation to high ground. In 2003 the first author was involved in a project to investigate the potential for reinforced concrete buildings to serve as vertical evacuation structures (Pacheco and Robertson, 2005). This study was part of a larger project to develop design guidelines for structures to serve as tsunami vertical evacuation sites (Yeh, et al., 2005). Primarily because of the experience of communities in low-lying regions such as Banda Aceh, with limited access to high ground near the shoreline, and short warning time for wave arrival, further attention was focused on the potential for vertical evacuation into buildings or other structures designed to resist tsunami loads. This led to the development of FEMA P-646: Guidelines for Design of Structures for Vertical Evacuation from Tsunamis by the Applied Technology Council (FEMA, 2012).

The Maule Tsunami of February 27, 2010, and the Tohoku Tsunami of March 11, 2011 caused tremendous damage to many coastal buildings, bridges and port facilities in Chile and Japan, respectively. However, a number of larger concrete and structural steel buildings survived with only non-structural damage, particularly if they had been designed for high seismic conditions. Field surveys following these events and analysis of survivor videos have provided a wealth of information on the tsunami flow characteristics and structural loading that need to be considered in the design of coastal buildings (Robertson, et al., 2012; Chock, et al., 2013).

Starting in February 2011, the ASCE Tsunami Loads and Effects Subcommittee worked for four and a half years to develop a new chapter for inclusion in the ASCE/SEI 7-16 Standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE, 2016). This new Chapter 6, Tsunami Loads and Effects, provides comprehensive provisions for design of coastal structures for tsunami loads, scour and related considerations.

ASCE/SEI 7-16 has since been adopted by the International Code Council for inclusion by reference in the 2018 version of the International Building Code (ICC, 2018) used throughout the United States of America. The tsunami design provisions will apply to all coastal communities in California, Oregon, Washington State, Alaska and Hawaii. A companion design manual has been developed by the first author to explain the new provisions and demonstrate their application to prototypical reinforced concrete and structural steel buildings in coastal communities in the Western USA (Robertson, 2019).

The ASCE/SEI 7-16 Tsunami Loads and Effects chapter is the first US national, consensus-based standard for tsunami resilience of critical and essential facilities, Tsunami Vertical Evacuation Refuge structures, and other multi-story building structures. The tsunami design provisions are consistent with the principles of probabilistic hazard analysis, tsunami physics, and fluid mechanics. They can be utilized in any tsunami-prone community once the probabilistic tsunami hazard for that location has been established.

The ASCE/SEI 7-16 standard currently only requires tsunami design for Tsunami Risk Category III (high occupancy buildings) and Tsunami Risk Category IV (Critical and essential facilities). Tsunami design is not required for the majority of buildings that fall into Tsunami Risk Category II, such as those used for residential, commercial and industrial purposes, for example. However, local jurisdictions are encouraged to consider requiring tsunami design for mid- to high-rise TRC II buildings so as to provide a "refuge-of-last-resort" for those who are not able to evacuate to high ground prior to tsunami arrival. The survival of taller buildings will also enhance the ability of the community to recover after the tsunami, thus increasing their resilience.

Prototypical Building Design

Two reinforced concrete prototypical building structures were developed for use in this study. The initial layout and structural configurations of these buildings were determined with the assistance of Gary Chock, president of Martin & Chock, Inc., a Honolulu structural engineering consulting company, to ensure that they represent realistic mid-rise office and residential buildings. These Risk Category II buildings were developed with sufficient height above grade (over 65 feet as suggested for severe tsunami hazard regions in the Commentary to ASCE 7 Chapter 6) to provide last-resort refuge for those caught in the tsunami inundation zone without sufficient time to evacuate to high ground, and to increase community resilience by ensuring that larger buildings remain intact except for non-structural components at the lower levels. The prototypical buildings were located near the shoreline and with their broad dimension facing the incoming tsunami flow. **Figure 1** and **Figure 2** show the Monterey, California, and Waikīkī, Hawaii, locations in Google Earth images, respectively.

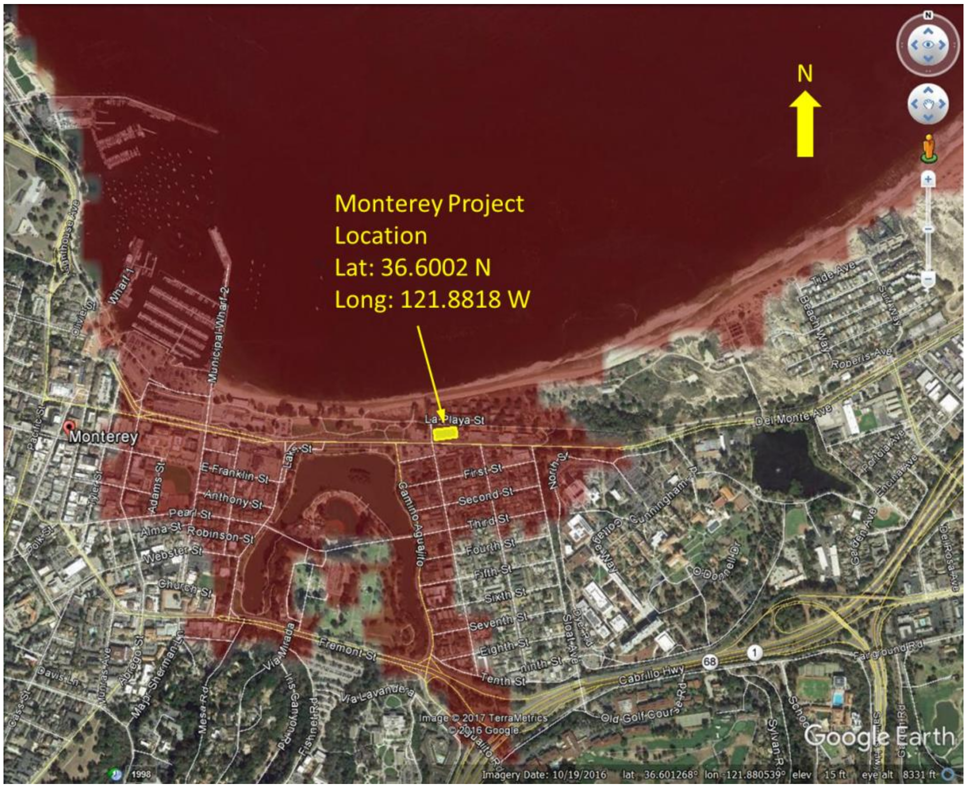

Figure 1: Monterey location for prototype buildings (Red shading indicates Tsunami Design Zone)

Figure 2: Waikīkī location for prototype buildings (Red shading indicates Tsunami Design Zone)

Reinforced Concrete Moment Resisting Frame Office Building

The prototype office building is a six story building consisting of reinforced concrete moment resisting frames (MRF), a flat plate post-tensioned concrete floor system, and interior gravity load columns. It was analyzed and designed for wind and seismic loading conditions specified by ASCE 7 for each site**.** **Figure 3** shows the floor plan and section for this building. This prototype building was analyzed and designed for the ASCE 7 specified wind and seismic conditions at three locations: Waikīkī, Hawaii; Hilo, Hawaii; and Monterey, California (Yokoyama and Robertson, 2014). Because of the similarity in seismic demand at Hilo and Monterey, the same building design is used at both locations. This prototype building also satisfies the seismic demand at Seaside, Oregon, so this was added as a fourth location. The Hilo, Monterey and Seaside buildings have special moment resisting frames on the building perimeter and at two transverse interior locations. The Waikīkī building has intermediate moment resisting frames at the same locations.

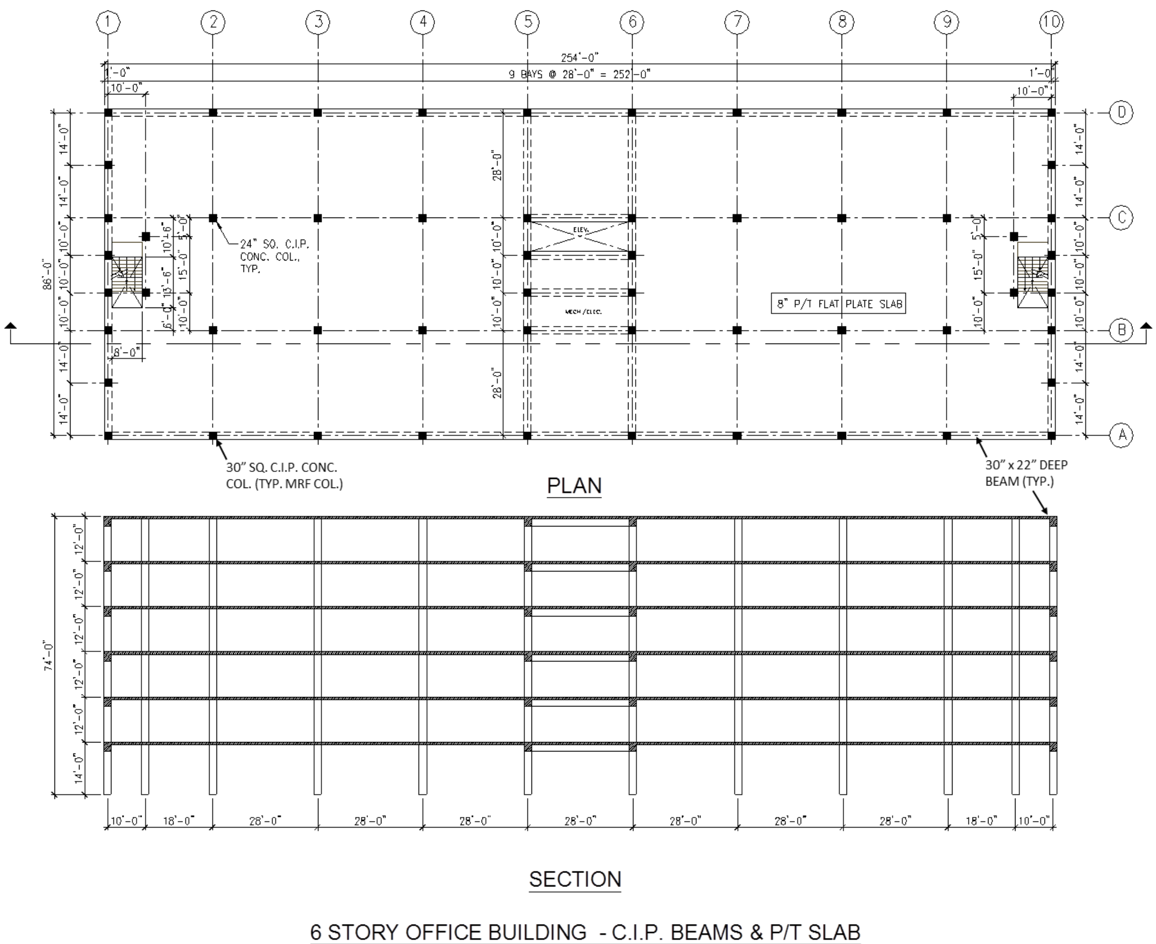

Figure 3: Prototype 6-Story Reinforced Concrete SMRF Office Building for Hilo, HI, Monterey, CA, and Seaside, OR - Plan and Section Views (The Waikīkī, HI, building has intermediate moment resisting frames at the same locations)

Reinforced Concrete Shear Wall Residential Building

The second prototype building is a seven story residential building consisting of reinforced concrete shear walls (SW) at elevator shafts and stairwells, a flat plate post-tensioned concrete floor system, and reinforced concrete gravity load columns**. Figure 4** shows the floor plan and section for the residential building. This prototype building was analyzed and designed for the wind and seismic conditions specified by ASCE/SEI 7-16 at three locations: Waikīkī, Hawaii; Hilo, Hawaii; and Monterey, California (Yokoyama and Robertson, 2014). Because of the similarity in seismic demand at Hilo and Monterey, the same building design is used at both locations. This prototype building also satisfies the seismic demand at Seaside, Oregon, so this was added as a fourth location. The residential buildings in Hilo, Monterey and Seaside have special reinforced concrete shear walls, while the Waikīkī building has ordinary reinforced concrete shear walls. It would be more common for the elevator shafts and stairwells to be located in the interior of the building, but here they are moved to the exterior so as to demonstrate the tsunami loading effects on exterior structural wall elements.

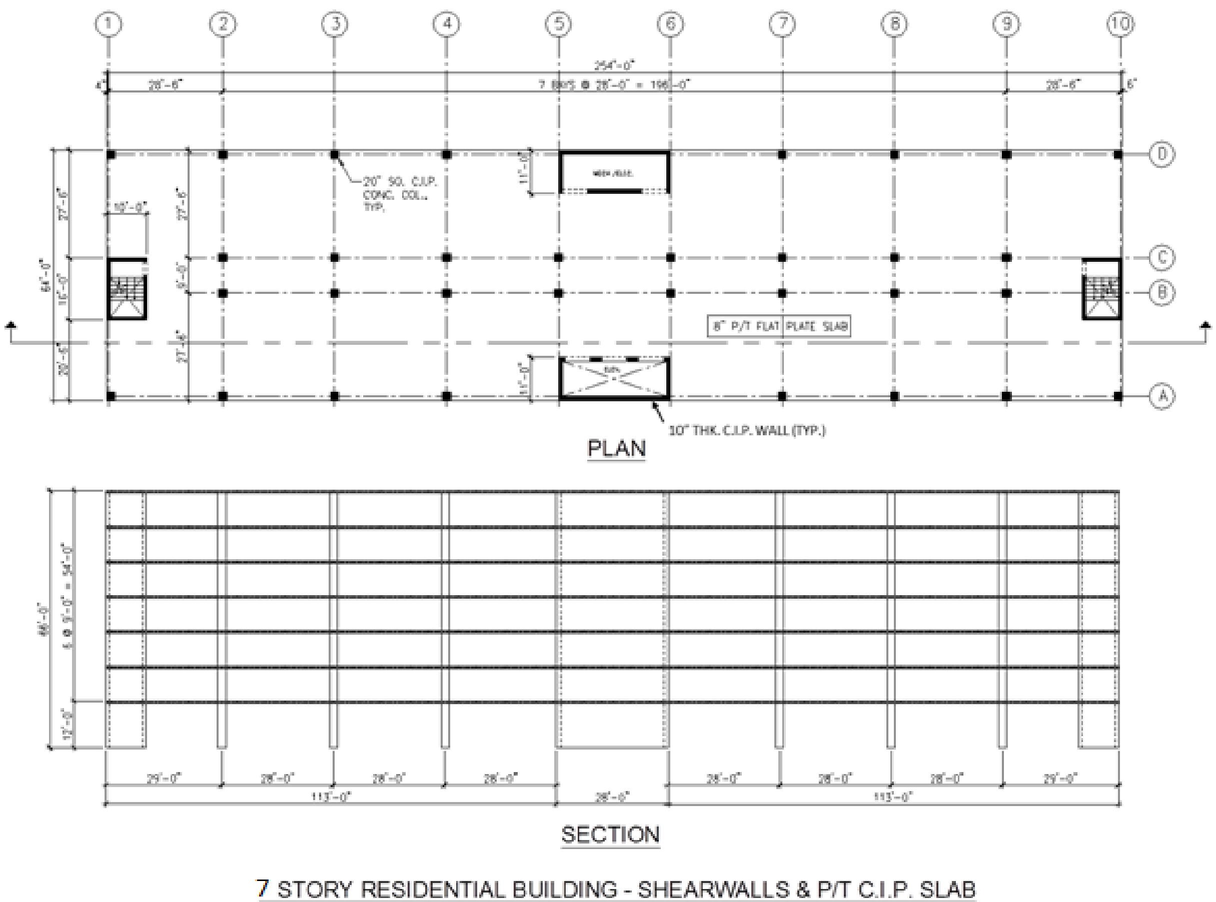

Figure 4: Prototype 7-Story Reinforced Concrete Shear Wall Residential Building - Plan and Section Views

Wind and Seismic Design

The prototype buildings were located in four coastal communities with varying wind, seismic and tsunami loading conditions. **Table 1** lists the locations with associated longitude and latitude, and design wind speeds according to ASCE 7-10. **Table 2** lists the seismic design criteria at each site assuming soil classification D for "stiff soil". The Waikīkī site has a high design wind speed but lower seismic demand than the other sites.

Each prototype building was analyzed and designed for the wind and seismic conditions at the two locations: Waikīkī, Hawaii, and Monterey, California (Yokoyama and Robertson, 2014). They were then evaluated for the tsunami loads required by ASCE 7-16 for these locations.

Table 1: Prototype building locations and associated design wind speeds

Location	Latitude	Longitude	Design Wind Speed (mph)
Monterey, CA	36.6002 N	121.8818 W	110
Seaside, OR	45.9948 N	123.9295 W	110
Hilo, HI	19.7209 N	155.0833 W	130
Waikīkī, HI	21.2755 N	157.8255 W	130

Table 2: Prototype building seismic design criteria

Location	Site Class	Seismic Design Category	S_S	S_1	S_{DS}	S_{D1}
Monterey, CA	D	D	1.513g	0.554g	1.009g	0.554g
Seaside, OR	D	D	1.332g	0.683g	0.888g	0.683g
Hilo, HI	D	D	1.500g	0.600g	1.000g	0.600g
Waikīkī, HI	D	C	0.579g	0.170g	0.516g	0.240g

Table 2 lists the seismic design parameters for all four sites considering site soil classification D. According to ASCE 7-10, all of the buildings fall into Seismic Design Category (SDC) D. However, the Honolulu Building Code amendments modify ASCE 7 Tables 11.6-1 and 11.6-2 so that Risk Category I and Risk Category II buildings can be designed as SDC C, if SDS<0.6g and SD1<0.25g (Honolulu City & County, 2012). Because of this local amendment, the Waikīkī location represents a lower level of seismic design and detailing than the other locations.

Tsunami Loading

The Seaside, Oregon, location will be used to demonstrate the tsunami loading on the office and residential prototype buildings. Only a brief overview of the tsunami load calculations will be included here. Additional detailed calculations are available in Appendix A. The building location is shown in **Figure 5**. A central topographic transect (C) is cut through the building site, oriented perpendicular to the shore line, represented by a straight line extending 500 meters either side of the transect. Two additional topographic transects are cut at ±22.5° as required by ASCE 7 and shown in **Figure 5**. These are referred to as the Clockwise (CW) and Counterclockwise (CCW) transects. These transects represent the range of tsunami flow directions that must be considered at the building site. The point where each transect crosses the runup line provides the runup elevation with respect to sea level datums Mean High Water and/or NAVD88 as shown in **Figure 5**. Sea level change was added to the runup elevations for an assumed fifty year project life cycle, as outlined in ASCE7 Commentary Section C6.5.3.

These topographic transects were used in the Energy Grade Line Analysis (EGLA) shown in **Figure 6** to determine the maximum flow depth (h_{max}) and velocity (u_{max}) at the site. The resulting values are provided in Table 2 of each Appendix for the respective building locations. More information on EGLA can be found in Kriebel et al. (2017) and Naito et al. (2016).

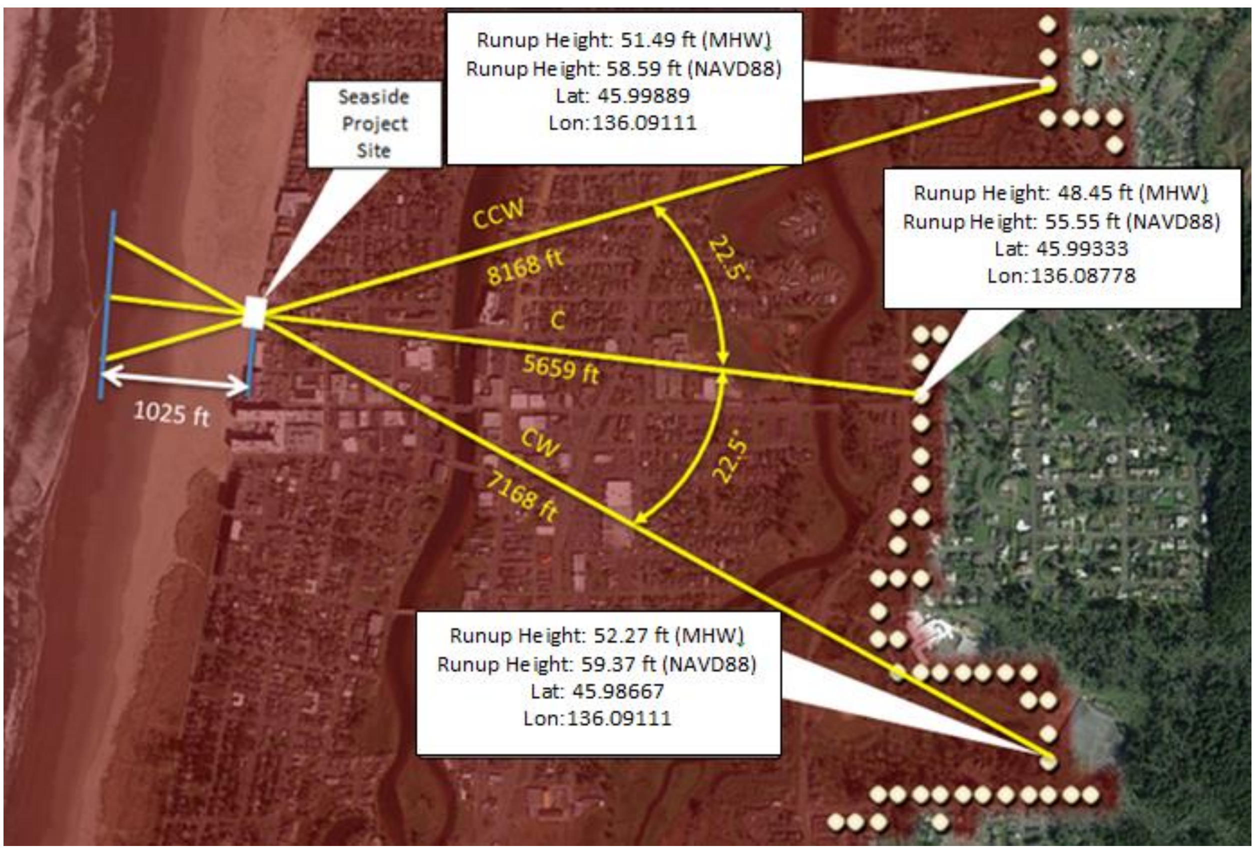

Figure 5: Seaside, Oregon, location showing three topographic transects and corresponding runup elevations.

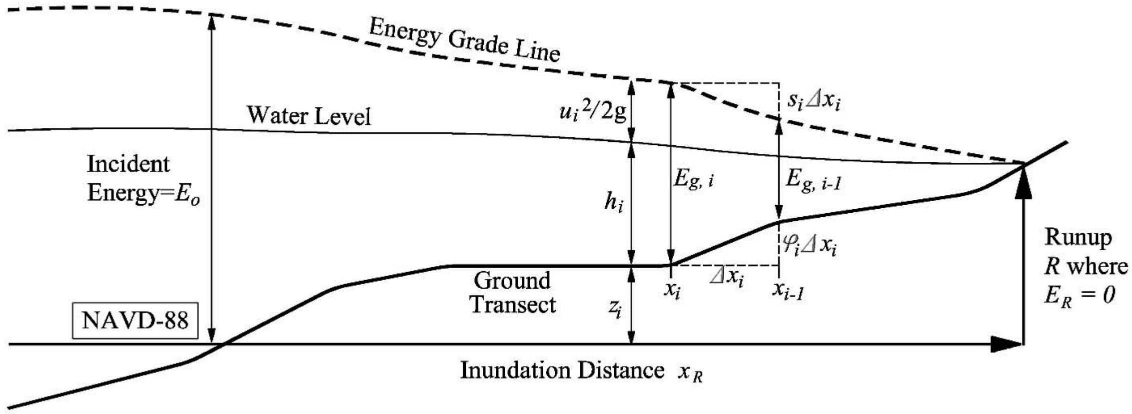

Figure 6: Schematic of Energy Grade Line Analysis (EGLA) (From ASCE 7-16)

In order to determine whether or not the EGLA must consider bore conditions at the shoreline, five criteria must be checked. Bore conditions must be considered if any of the following is met:

- The prevailing nearshore bathymetric slope is 1⁄100 or milder.
- Shallow fringing reefs or other similar step discontinuities occur in the nearshore bathymetry.
- Where historically documented.
- As described in the recognized literature, or
- As determined by a site-specific inundation analysis.

The controlling criterion for Seaside, Oregon, is the average nearshore bathymetry which is much less than the 1/100 limit as shown in **Figure 7**. The distance to the off-shore wave heights, which are plotted at a bathymetric depth of 100 meters, provides a simple calculation of the average near shore bathymetric slope.

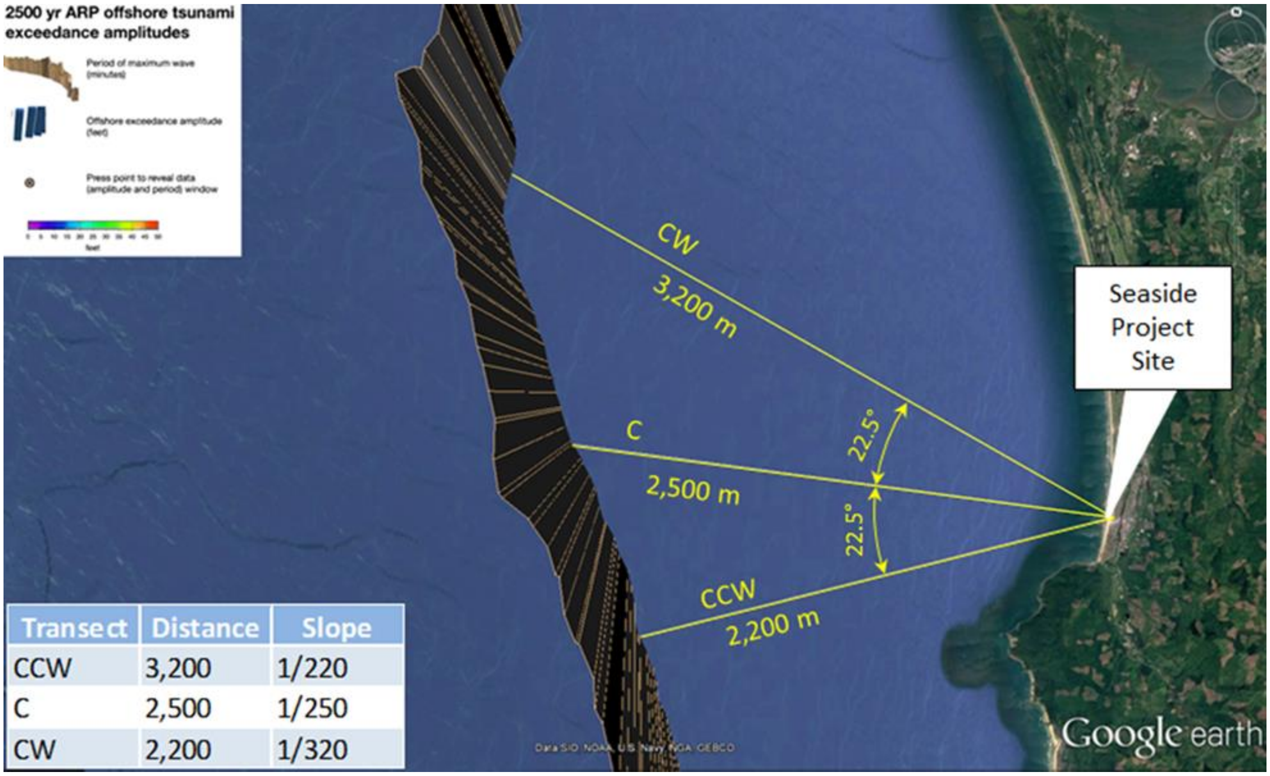

Figure 7: Determination of incoming wave conditions at Seaside, Oregon.

Office Building Tsunami Design

The office building at the Seaside, Oregon, location consists of an exterior moment resisting frame with two interior transverse MRFs. The remaining interior columns are gravity load only, so do not contribute to the lateral framing system.

Overall Building loading

According to ASCE 7-16, three load cases must be evaluated in order to determine the overall tsunami hydrodynamic loading applied to the building. Load Case 1 is a check for buoyancy and associated hydrodynamic drag. The flow depth for Load Case 1 is the smallest of 1) ground floor story height, 2) height to the top of the first story windows, or 3) the maximum flow depth, h_{max}. For this example the top of the first floor windows is 8ft so for LC1 h= 8'. Load Case 2 considers the maximum flow velocity, u_{max} , which is assumed to occur when the flow depth is (2/3)h_{max} . Load Case3 considers the maximum inundation depth, h_{max} , with a flow velocity of (1/3)u_{max} .

For each load case the hydrodynamic drag (F_{dx}) is determined using the following equation;

$$F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) \qquad \text{(ASCE 7-16, Eqn. 6.10-2)}$$

where:

ρ_s = Minimum fluid mass density for design hydrodynamic loads (1.1 x 2.0 = 2.2 slugs/cuft),

I_{tsu} = Importance factor for tsunami forces to account for additional uncertainty in estimated parameters = 1.0 for Tsunami Risk Category II buildings (ASCE 7-16, Table 6.8-1).

C_d = Drag coefficient based on quasi-steady state forces = 1.4575 based on B/h_{sx} = 254/8 = 31.8 (ASCE 7-16, Table 6.10-1),

C_{cx} = Proportion of closure coefficient from ASCE 7-16 Eqn. 6.10-3, but not less than 0.7,

B = overall width of building (254 feet for both the office and residential buildings),

h = Tsunami inundation depth above grade plane at the structure, and

u = Tsunami flow velocity.

For all three load cases, C_{cx} is lower than 0.7, so the default value for regular structures of 0.7 is used for all three. As an example, the following computation is for the Seaside example LC2 with a flow depth of 2/3 h_{max} = 20.93 ft.

$$C_{cx} = \frac{\sum(A_{col} + A_{wall}) + 1.5A_{beam}}{Bh_{sx}} = \frac{\sum((1764 + 648) + 0) + 1.5 \times 508}{254' \times 20.93'} = 0.597 < 0.7$$

A_{col} = Total projected area of the columns inundated (ft^2)

A_{wall} = Total projected area of the walls inundated (ft^2)

A_{beam} = Total projected area of the beams inundated (ft^2)

h_{sx} = Tsunami inundation depth above grade plane for the given load case (ft)

The resulting overall hydrodynamic drag on the building at the Seaside location for each of the three load cases are listed in **Table 3**.

Table 3: Overall building hydrodynamic drag for Seaside location

Load Case	Flow Depth (ft)	Flow Velocity (ft/sec)	Hydrodynamic Drag (kips)	Distributed Load (kips/ft)
LC1	8	26.54	2295	287
LC2	20.93	37.92	7369	352
LC3	31.4	12.64	1226	39

Figure 8 and **Figure 9** show the overall building lateral hydrodynamic load for load cases 2 and 3, respectively, for the Seaside location. The lateral load is shown both as a uniformly distributed load (kip/ft) and as concentrated loads (kips) at each floor level, determined according to the height tributary to each floor, including the ground level.

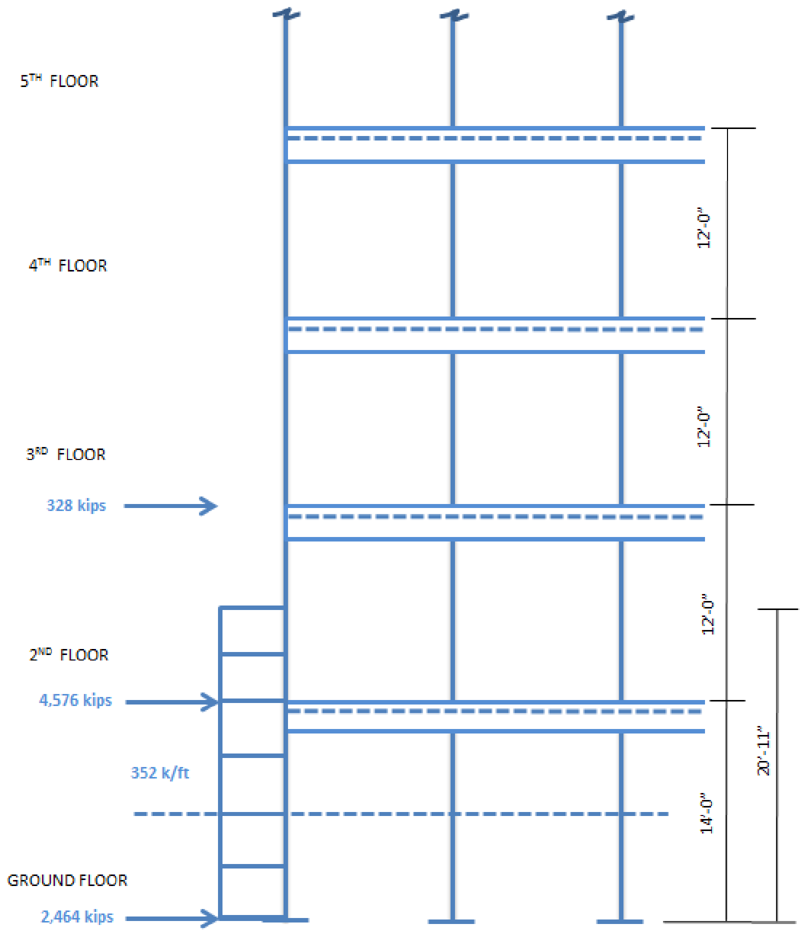

Figure 8: Overall building lateral hydrodynamic load for Load Case 2 at Seaside location.

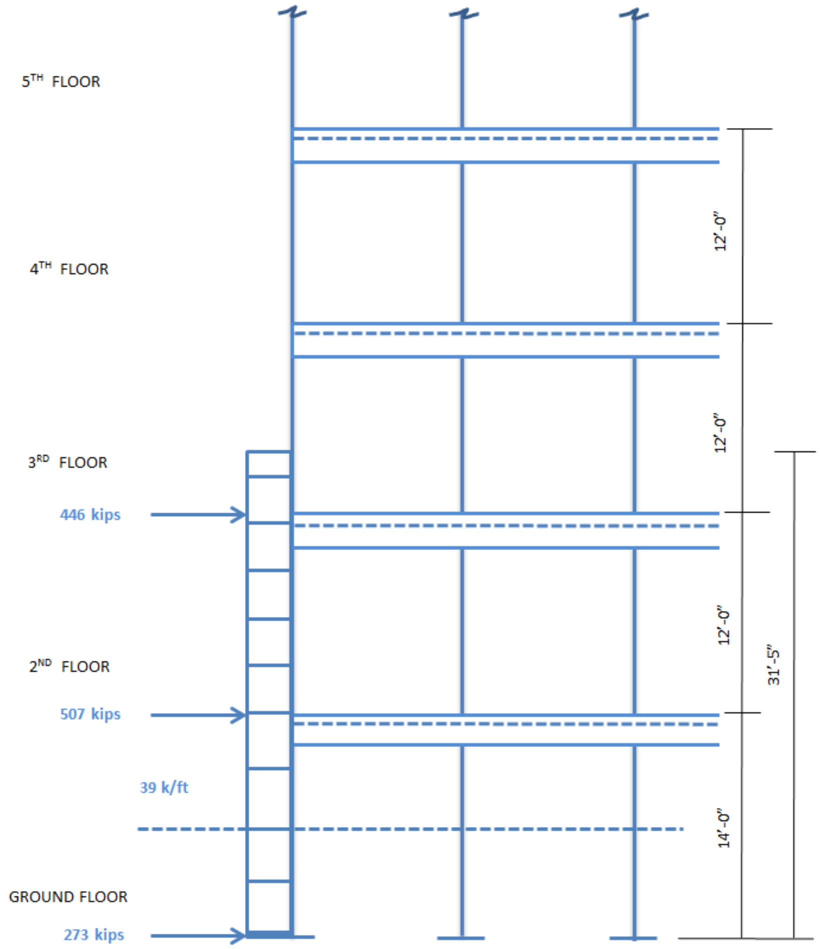

Figure 9: Overall building lateral hydrodynamic load for Load Case 3 at Seaside location.

For buildings that are designed for Seismic Design Category D, E or F, ASCE 7-16 allows the design to utilize portion of the seismic overstrength to resist the tsunami loads. Section 6.8.3.4 states that;

> *"To evaluate the capacity of the structural system at the Life Safety Structural Performance Level to resist the lateral force effects of the design tsunami event for Seismic Design Category D, E, or F, it is permitted to use the value of 0.75 times the required Horizontal Seismic Load Effect, E_{mh}, which includes the system's overstrength factor, Ω_0, as defined in Chapter 12 of this standard."*

This implies that if the tsunami base shear, $V_{TSU} < 0.75\ \Omega_o\ E_h$, then the seismic structural system is adequate to resist the overall building tsunami loads. If V_{TSU} exceeds this value, then E_h must be increased until the check is met. In other words, in order to satisfy the overall tsunami demand on the building, V_{TSU}, the structural system must be capable of resisting a seismic base shear of:

$$E_h = \frac{V_{TSU}}{0.75\Omega_o}$$

In order to resist the tsunami loads, the lateral force resisting system must be designed for a seismic base shear of at least $E_h = V_{TSU}/(0.75\ \Omega_o)$, where V_{TSU} is the tsunami base shear and Ω_o is the seismic overstrength factor.

An ETABS computer model of the moment resisting frame building was used to analyze the structural response to the overall building tsunami loads (**Figure 10**). The modified seismic base shear, E_h, was applied as concentrated loads at each floor level, distributed in accordance with the seismic lateral load requirements of ASCE 7-16, and located at the center of the building face. The resulting axial load, shear and bending moments at the base of each of the MRF columns for the Load Case 2 analysis are shown in **Figure 11**. These systemic loads in each column will be combined with the individual component loads on the column determined from individual member hydrodynamic drag and impact loading. The columns at all inundated levels will then be evaluated for these combined tsunami loads and modified as required. Because the MRF beams are integral with the floor slabs at each level, the hydrodynamic lateral load on the beams will be transferred directly to the slab, without causing lateral moments in the beams.

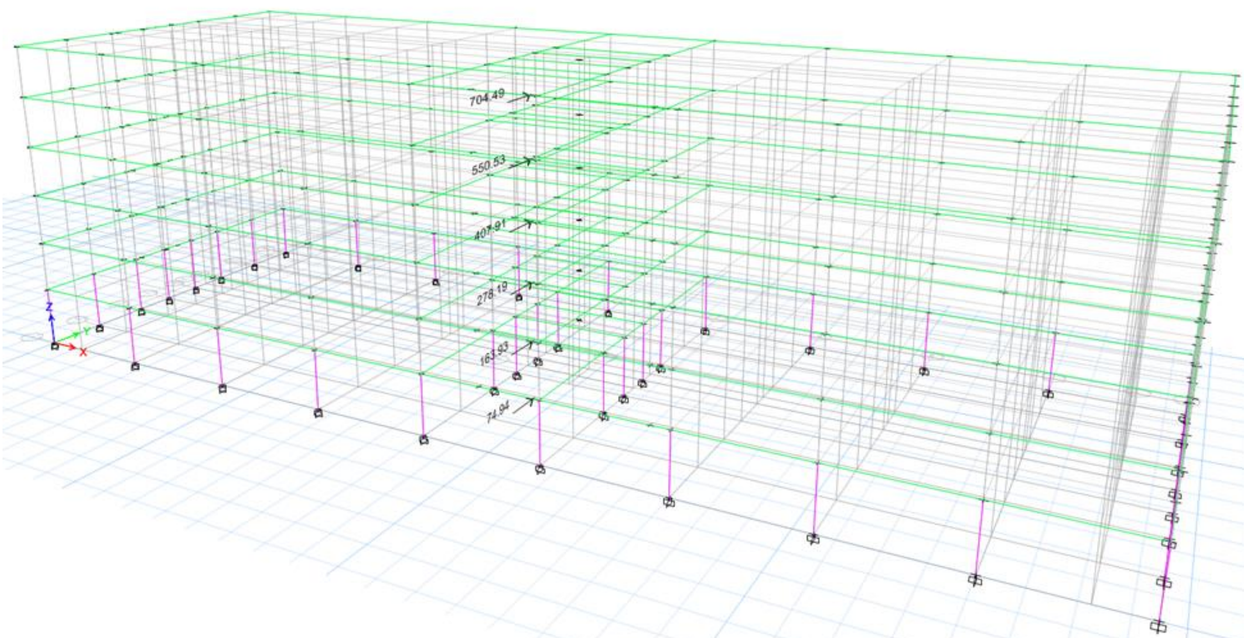

Figure 10: ETABS computer model of Moment Resisting Frame office building (with seismic lateral loads shown).

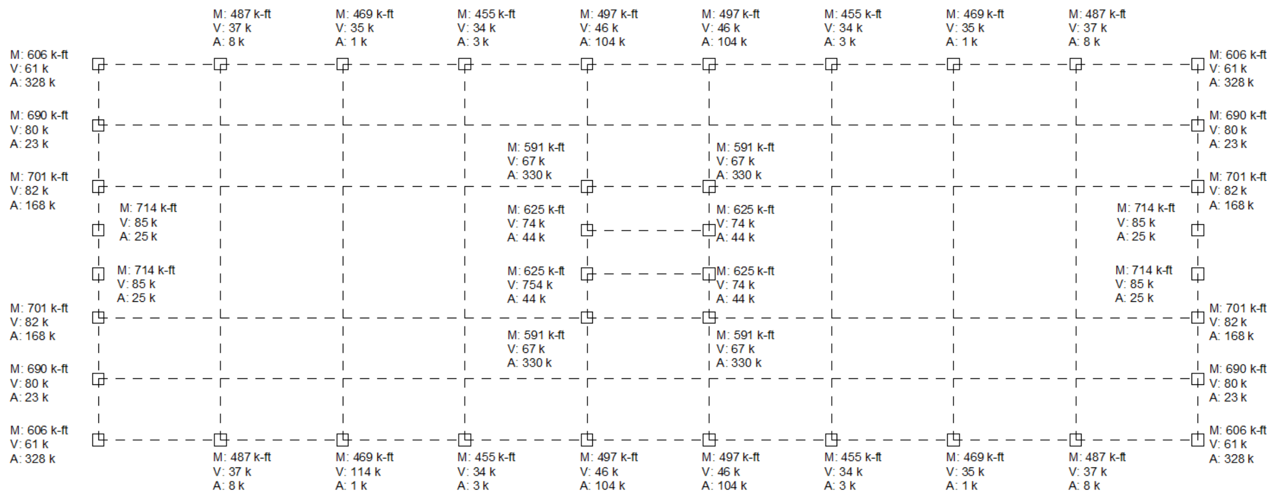

Figure 11: ETABS analysis results for first floor columns of the MRF office building due to overall building lateral hydrodynamic load for Load Case 2 at the Seaside location.

Component Hydrodynamic Loads

In addition to resisting a portion of the overall building lateral tsunami loads, each of the components of the MRF must also resist the individual component loads defined by ASCE 7-16. Members that are not part of the MRF need only resist the individual component loads. These loads are due to either hydrodynamic drag or debris impact on the individual member. ASCE 7-16 does not require that hydrodynamic drag and debris impact be considered to happen simultaneously because of the low probability that the maximum of each type of load will occur at the same time.

The component hydrodynamic load (F_d) is calculated using the drag force on components given by **ASCE 7-16 Eqn. 6.10-4**. Exterior columns must resist the drag force assuming debris damming equivalent to C_{cx} times the column tributary width. The resulting distributed lateral load will then be applied to the column to determine the resulting shears and bending moments.

The following is a sample calculation for a typical MRF exterior column with a tributary width (column spacing) of 28 feet exposed to Load Case 2:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

$$\therefore F_d = \frac{1}{2} * 2.2\frac{Slug}{ft^3} * 1 * 2 * 19.6'\,(20.93' * (37.92\frac{ft}{sec.})^2)$$

$$\therefore F_d = 1{,}298\ Kips$$

ρ_s = Minimum fluid mass density for design hydrodynamic loads (1.1 x 2.0 = 2.2 slugs/cuft)

I_{tsu} = Importance Factor for tsunami forces to account for additional uncertainty in estimated parameters = 1.0 for TRC II buildings (**Table 6.8-1**)

C_d = Drag coefficient based on quasi-steady forces (Debris damming, C_d = 2)

b = effected loading area = 28' x 0.7 =19.6' (tributary width multiplied by the closure coefficient from ASCE 7-16 Section 6.8.7)

h_e = Tsunami inundation height for the individual element (20.93')

u = Tsunami flow velocity (37.92 $\frac{ft}{sec.}$ for Load Case 2 at Seaside location)

For the typical MRF exterior column exposed to Load Case 2, the distributed load up the submerged height of the column is F_d/h_e = 1298/20.93 = 62 k/ft. This load is applied to the exterior column as shown in **Figure 12**. The column is modeled as fixed at the base and pin-supported at each slab level. The resulting shear force and bending moment distributions are shown in **Figure 12**.

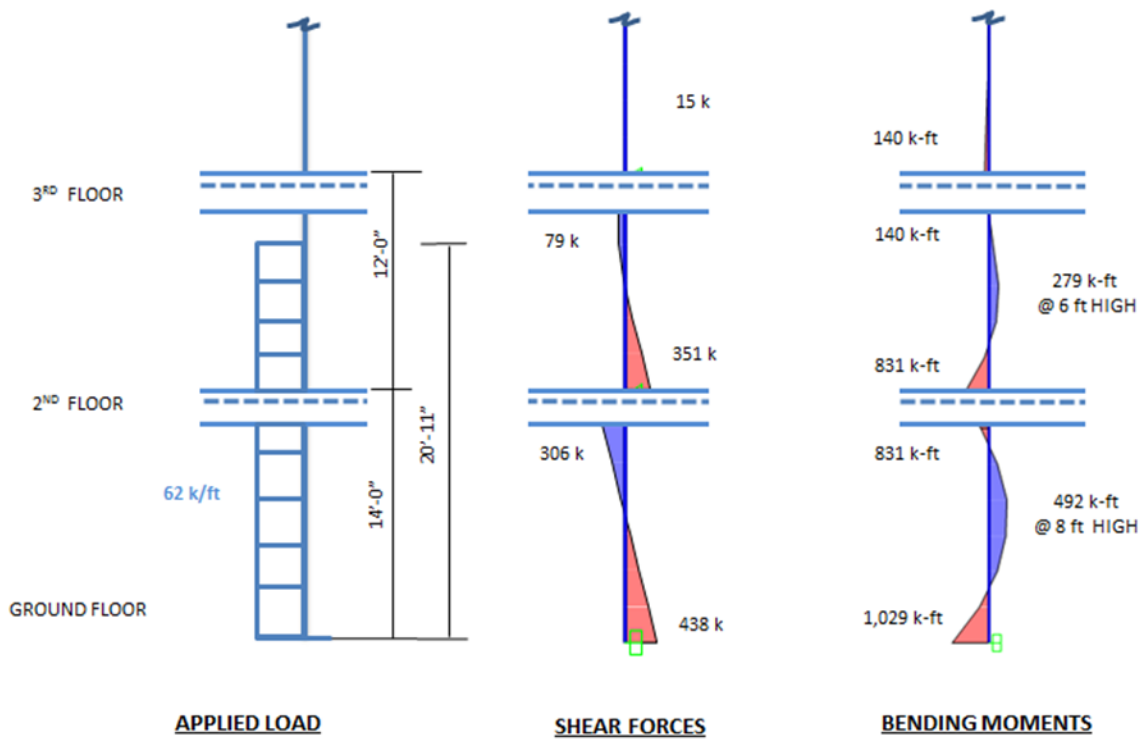

Figure 12: Component hydrodynamic lateral load applied to an exterior MRF column, with associated shear force and bending moment diagrams.

Debris impact loads

The prototypical buildings are not located in a large debris influence area (near port, harbor or shipping container storage yard). The debris impacts that must be considered for this building are vehicles, rolling boulders and wood logs and poles. The controlling impact load is due to a log or pole.

As shown in Appendix A, the log impact for Load Case 2 at the Seaside site is governed by the alternative simplified debris impact static load given in ASCE 7-16 Section 6.11.1. In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330C_0I_{tsu} = 330 \times 0.65 \times 1.0 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force will be reduced to 50%, or 107.25 kips. This load must be applied to the exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with hydrodynamic loads on the column, but it must be combined with systemic loads if the member is part of the lateral force resisting system. In the event that this load exceeds the column capacity, the column can be designed for the static impact loads, or a detailed debris impact analysis can be performed. Debris impact loads are not applied to interior columns.

For maximum bending moment in the column, the log impact load is applied at the mid height of the column. For maximum shear force in the column, the log impact load is applied at a distance *d* from each end of the column, where *d* is the effective depth of the column cross section. Because the ends of a seismic MRF column have additional ties for seismic ductility requirements, it is also necessary to check the shear in the column section outside the ductile end regions of the column. This implies application of the impact load at a distance ($d + h_c$) from the ends of the column, where h_c is the length of the column end hinging region, typically equal to the maximum column cross-section dimension. **Figure 14** and **Figure 15** show the impact load applied a distance *d* and ($d + h_c$) away from the top of the first floor column. This load would also be applied at similar locations at the bottom of the first floor column, and at similar locations on all other floors below the maximum flow depth, h_{max}. **Figure 16** shows application of the impact load at mid-height of the column.

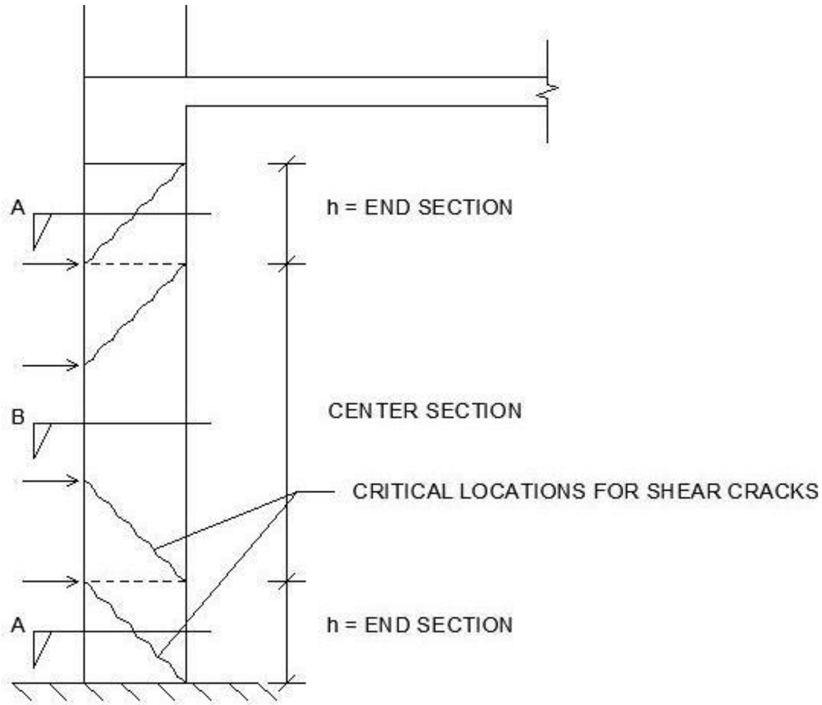

Figure 13: Critical impact load locations for shear force in an exterior MRF column.

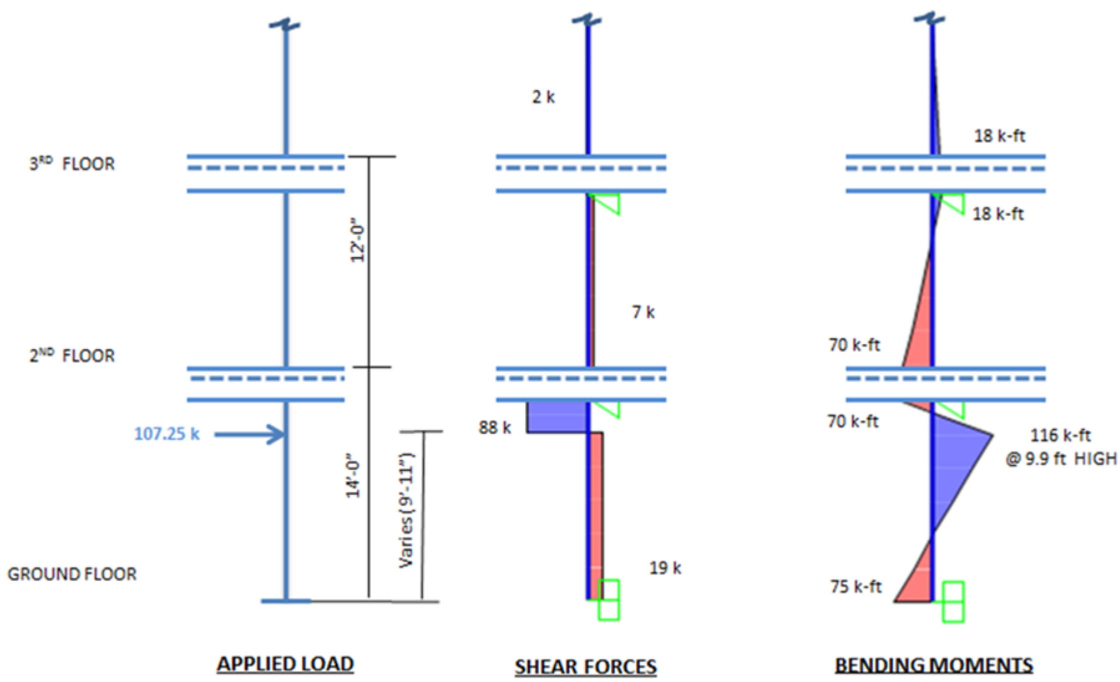

Figure 14: Impact load applied at "d" away from the top of the column on the ground floor level

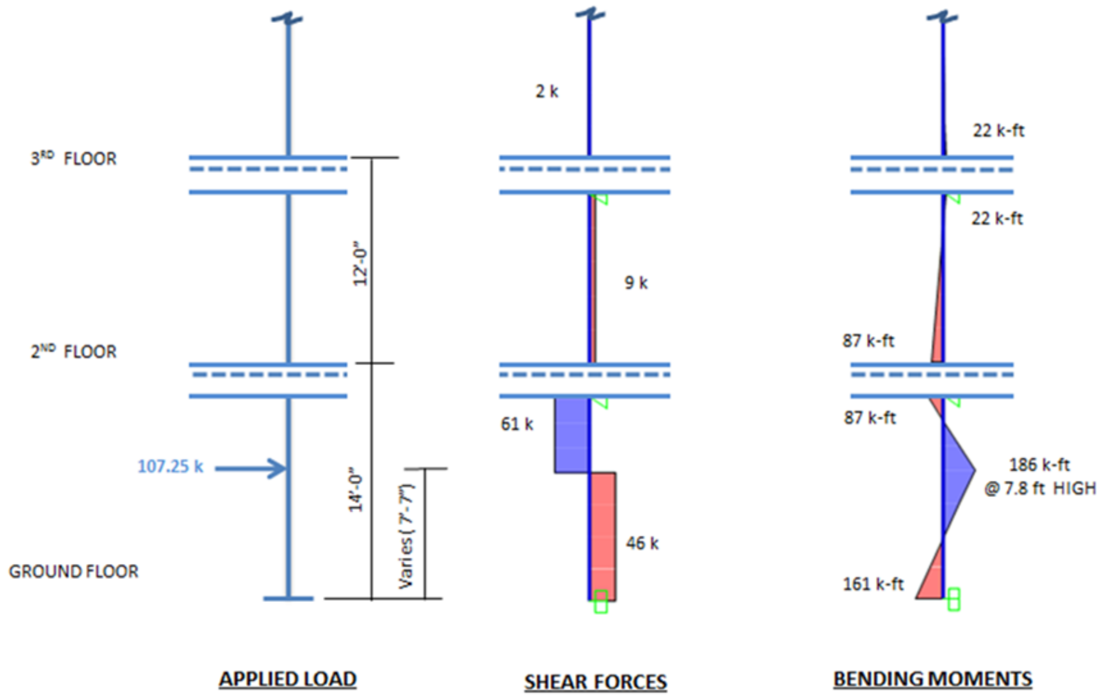

Figure 15: Impact load applied at "d + h_c" away from the top of the column on the ground floor level

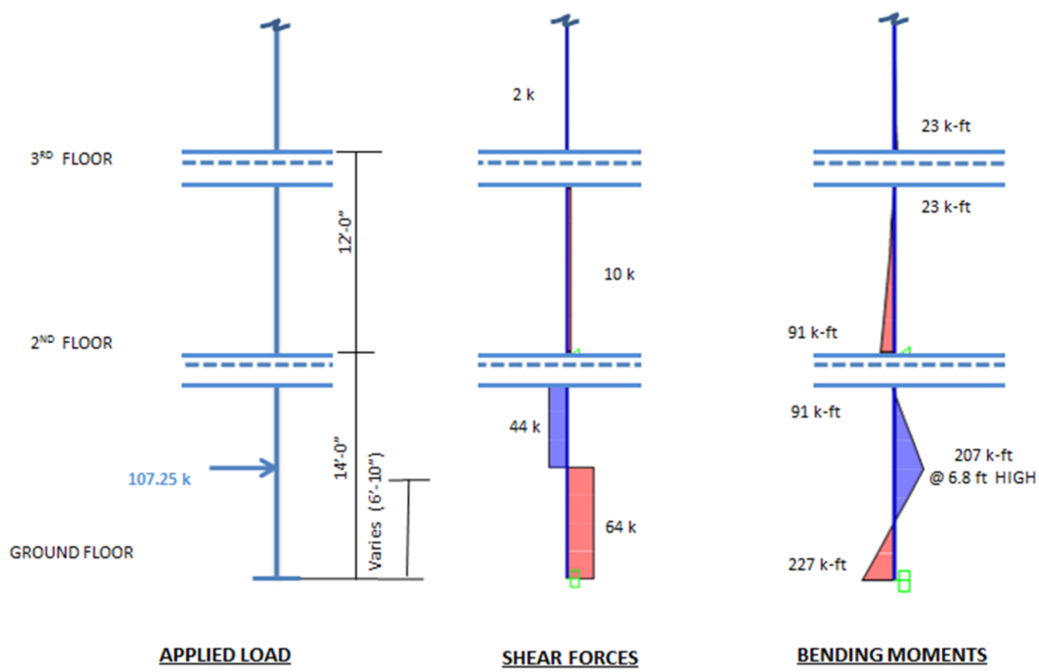

Figure 16: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

Tsunami Design Implications

Exterior Columns

Because the exterior columns are all part of the lateral load resisting MRF for this building, the member forces caused by the overall building lateral load must be combined with the member forces caused by either the component hydrodynamic force or the debris loads. The required load combinations are given by ASCE 7-16 Section 6.8.3.3 as (1.2D + F_{tsu} + 0.5L) and (0.9D + F_{tsu}).

Table 4 summarizes the maximum axial load, bending moment and shear forces for all inundated exterior MRF columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and log impact (Impact). In addition, because all of the exterior columns are part of the LFRS, **Table 4** also lists the maximum axial load, bending moment and shear forces determined by the ETABS analysis for the modified base shear (Overall). These "Overall" systemic forces are then combined with the controlling component forces (either "Hydro" or "Impact") to obtain the "Combined" forces.

The original column designs are then evaluated for these load combinations and modified if necessary.

Table 4: Load Combinations for typical exterior MRF column at Seaside office building

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
1029	502.3	307	162	1.2D+Ftsu+0.5L (Hydro)
1029	355.5	307	162	0.9D+Ftsu (Hydro)
227	502.3	88	61	1.2D+Ftsu+0.5L (Impact)
227	355.5	88	61	0.9D+Ftsu (Impact)
701	334.25	85	85	1.2D+Ftsu+0.5L (Overall)
701	187.5	85	85	0.9D+Ftsu (Overall)
1526	398.25	392	247	1.2D+Ftsu+0.5L (Combined)
1526	251.5	392	247	0.9D+Ftsu (Combined)
Floor 2				
831	418.5	220	75	1.2D+Ftsu+0.5L (Hydro)
831	296.3	220	75	0.9D+Ftsu (Hydro)
221	418.5	88	60	1.2D+Ftsu+0.5L (Impact)
221	296.3	88	60	0.9D+Ftsu (Impact)
651	295.5	113	113	1.2D+Ftsu+0.5L (Overall)
651	173.25	113	113	0.9D+Ftsu (Overall)
1007	335.5	333	188	1.2D+Ftsu+0.5L (Combined)
1007	213.25	333	188	0.9D+Ftsu (Combined)
Floor 3				
140	334.8	15	15	1.2D+Ftsu+0.5L (Hydro)
140	237	15	15	0.9D+Ftsu (Hydro)
221	334.8	87	60	1.2D+Ftsu+0.5L (Impact)
221	237	87	60	0.9D+Ftsu (Impact)
117	330.8	20	20	1.2D+Ftsu+0.5L (Overall)
117	233	20	20	0.9D+Ftsu (Overall)
328	330.8	107	80	1.2D+Ftsu+0.5L (Combined)
328	233	107	80	0.9D+Ftsu (Combined)
Floor 4				
35	251.1	4	4	1.2D+Ftsu+0.5L (Hydro)
35	177.8	4	4	0.9D+Ftsu (Hydro)
98	251.1	10	10	1.2D+Ftsu+0.5L (Impact)
98	177.8	10	10	0.9D+Ftsu (Impact)
Floor 5				
9	167.4	1	1	1.2D+Ftsu+0.5L (Hydro)
9	118.5	1	1	0.9D+Ftsu (Hydro)
24	167.4	3	3	1.2D+Ftsu+0.5L (Impact)
24	118.5	3	3	0.9D+Ftsu (Impact)
Floor 6				
2	83.7	0	0	1.2D+Ftsu+0.5L (Hydro)
2	59.3	0	0	0.9D+Ftsu (Hydro)
6	83.7	1	1	1.2D+Ftsu+0.5L (Impact)
6	59.3	1	1	0.9D+Ftsu (Impact)

Exterior Column Bending Design

Figure 17 shows interaction diagrams for a typical exterior MRF column including the tsunami load combinations. The blue solid line (Original Column Design Strength) represents the design strength for the original column. The green dashed line (New Column Design Strength) represents the design strength needed if one were to take into account only the component hydrodynamic and impact loads. The dotted red line (New Overall Column Design Strength) represents the design strength needed for taking into account only the overall building forces for the column. The orange dot-dashed line (New Combined Column Design Strength) represents the design strength needed for the overall loading combined with the component hydrodynamic or impact loads for the column. This series of plots is shown in Appendix A for all affected floor levels of the Seaside office building**. Figure 18** shows the interaction diagram for the combined forces with the controlling load combination for each exterior column at the ground floor level.

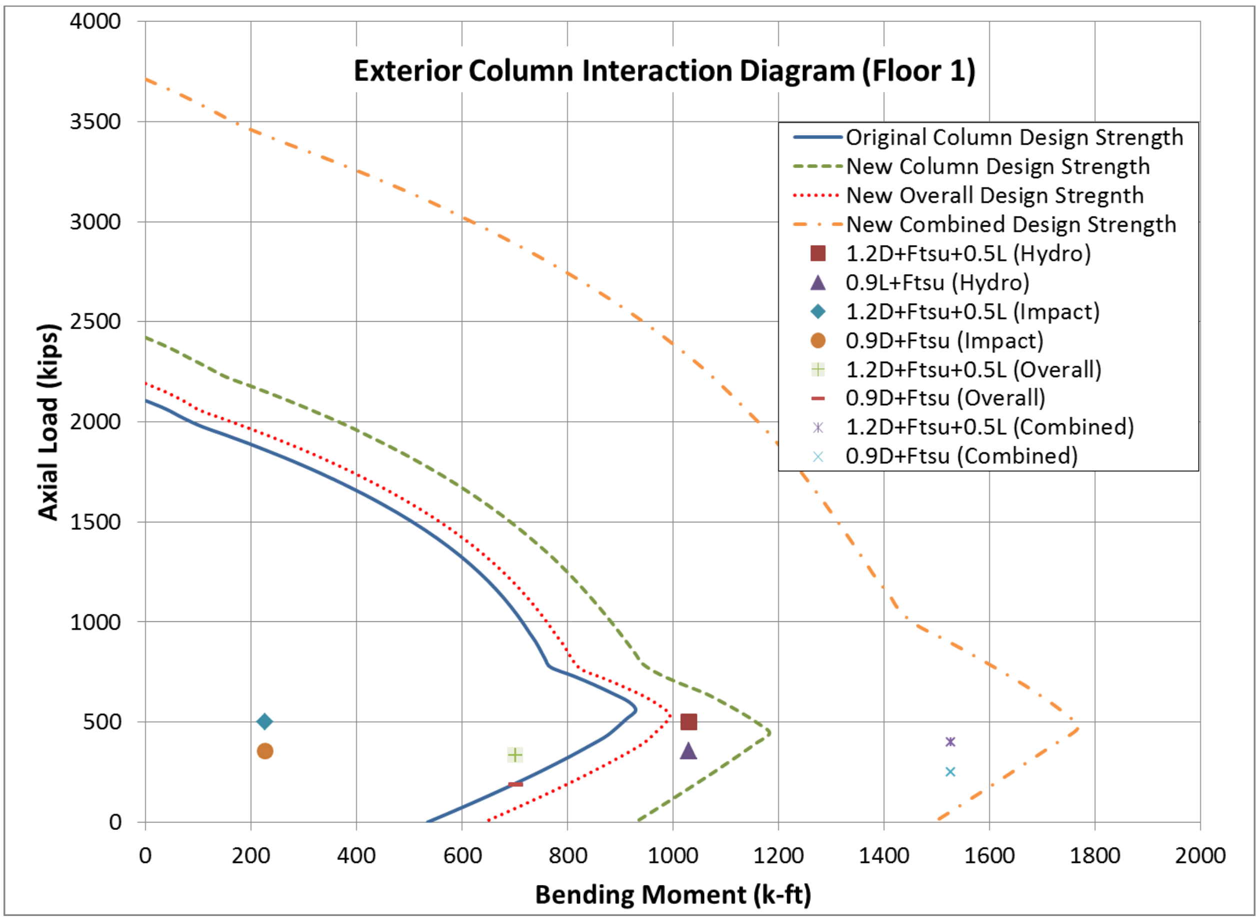

Figure 17: Sequence of interaction diagrams for typical ground floor exterior column showing tsunami load combinations

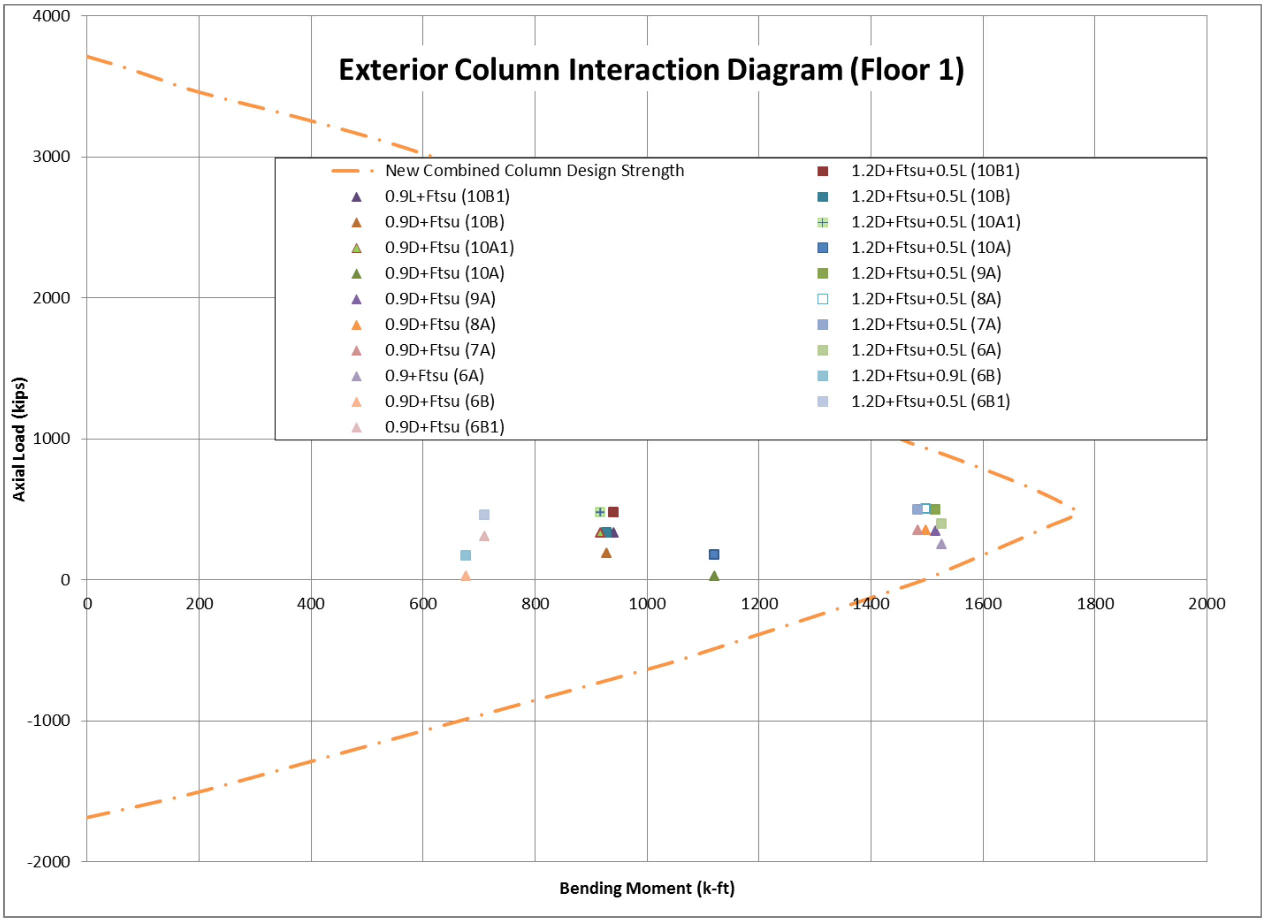

Figure 18: Interaction diagram for all ground floor exterior columns showing all combined tsunami load combinations

Based on the blue line interaction curve in **Figure 17**, the existing MRF exterior columns are not adequate to resist the combined tsunami lateral loads. The existing column cross-sections at the end and center sections are shown in **Figure 19** and **Figure 20**, respectively.

In order to utilize the same column formwork at all levels, it was preferable to maintain the column size at 28” square, so the reinforcement was increased to satisfy the tsunami design loads. The resulting column cross-sections at end and center of the column required to meet tsunami loads are shown in **Figure 21** and **Figure 22**, respectively.

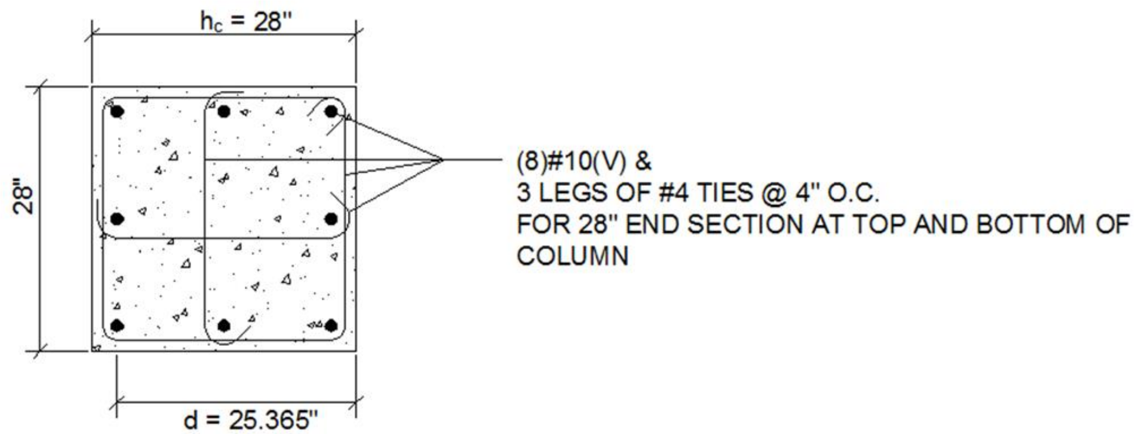

Figure 19: Existing exterior MRF column cross-section at end of column at ground floor level based on SDC D design.

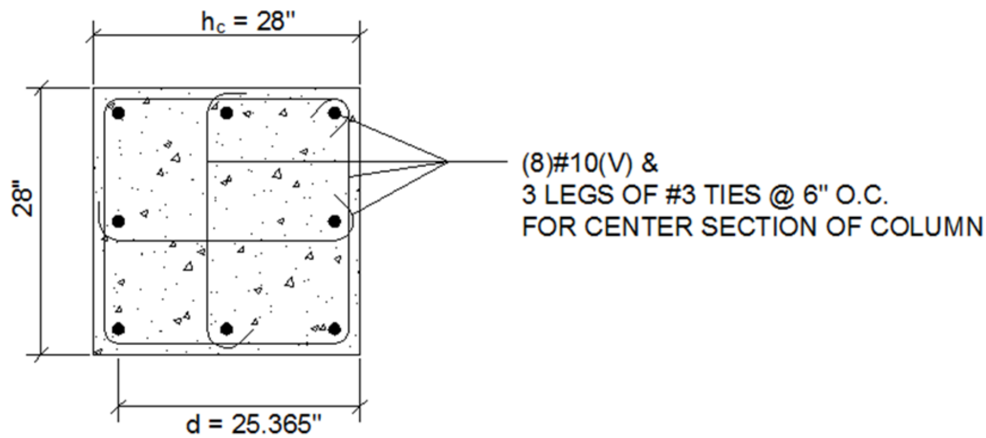

Figure 20: Existing exterior MRF column cross-section at center of column at ground floor level based on SDC D design.

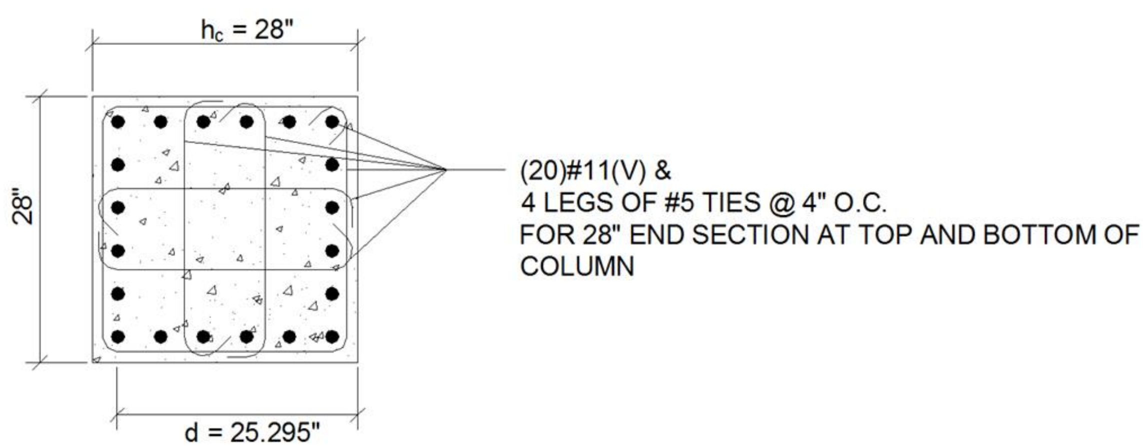

Figure 21: New exterior column cross-section at end section of column at ground floor level based on tsunami design.

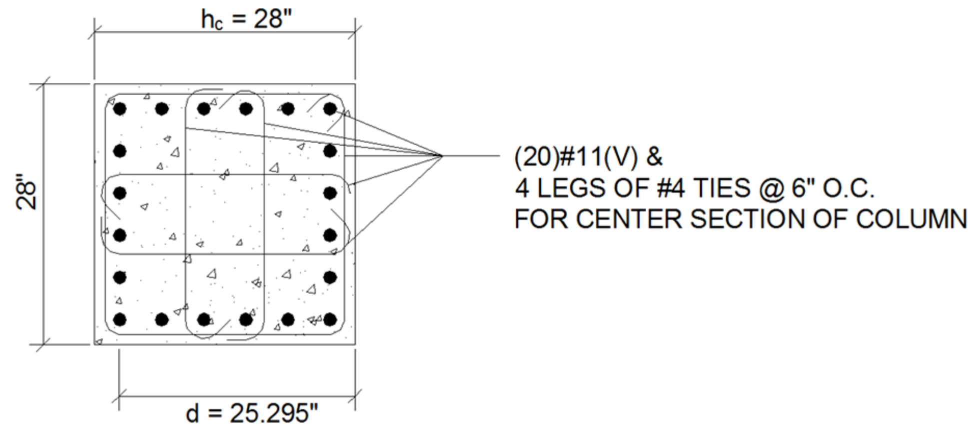

Figure 22: New exterior column cross-section at center section of column at ground floor level based on tsunami design.

Exterior Column Shear Design

The new columns must also be checked for shear. The shear forces for the combined loading were determined by adding the maximum shear load of either the hydrodynamic loading or the impact loading with the overall systemic shear at both "*d*" and "$d + h_c$" locations as shown in **Table 4**. The shear checks are then performed according to ACI 318 as follows:

In order to satisfy the ACI 318 shear design, the factored shear force, V_u, must not exceed the design shear strength, ϕV_n, where:

$$ØV_n = Ø(V_C + V_S)$$

and $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{6{,}000}\left(1 + \frac{251{,}500}{2{,}000\times28\times28}\right)28 \times 25.295/1{,}000 = 127$ kips

In the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.31)\times60{,}000\times25.295}{4\times1{,}000} = 470$ kips

$$V_S = \frac{A_v f_y d}{s} = 470 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{6{,}000} \times 28 \times 25.295 = 439 \text{ kips} \therefore \text{use } 439 \text{ kips}$$

In the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times25.295}{6\times1{,}000} = 202$ kips

$$V_S = \frac{A_v f_y d}{s} = 202 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{6{,}000} \times 28 \times 25.295 = 439 \text{ kips} \therefore \text{use } 202 \text{ kips}$$

therefore in the end sections, $ØV_n = 0.75\ (127 + 439) = 425$ k

therefore in the center sections, $ØV_n = 0.75\ (127 + 202) = 247$ k

At d: $V_u = 392$ k < $\phi V_n = 425$ k, therefore the column is adequate for shear at the edge.

At $d + h_c$: $V_u = 247$ k ≤ $\phi V_n = 247$ k, therefore the column is adequate for shear at the center.

.

Interior Columns

For interior columns the process is similar except that impact loads do not apply, and the component hydrodynamic loading only considers the column width without any debris damming. As a result, for Load Case 2, the interior 24”x24” gravity load columns experience a lateral load given by;

$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} * 2.2\frac{Slug}{ft^3} * 1 * 2 * 2'\ (20.93' * (37.92\frac{ft}{sec.})^2)$ = 132.4 Kips

ρ_s = Minimum fluid mass density for design hydrodynamic loads (1.1 x 2.0 = 2.2 slugs/cuft)

I_{tsu} = Importance Factor for tsunami forces to account for additional uncertainty in estimated parameters 1.0 (**Table 6.8-1** – TRC II)

C_d = Drag coefficient based on quasi-steady forces (Rectangular columns C_d = 2 (**Table 6.10-2**))

b = column width = 2’

h_e = Tsunami inundation height of an individual element (20.93’)

u = Tsunami flow velocity ($37.92\frac{ft}{sec.}$)

For the typical interior gravity load column exposed to Load Case 2, the distributed load up the submerged height of the column is F_d/h_e = 132.4/20.93 = 6.3 k/ft. This load is applied to the interior column as shown in **Figure 23**. The column is modeled as fixed at the base and pin-supported at each slab level. The resulting shear force and bending moment distributions are shown in **Figure 23**.

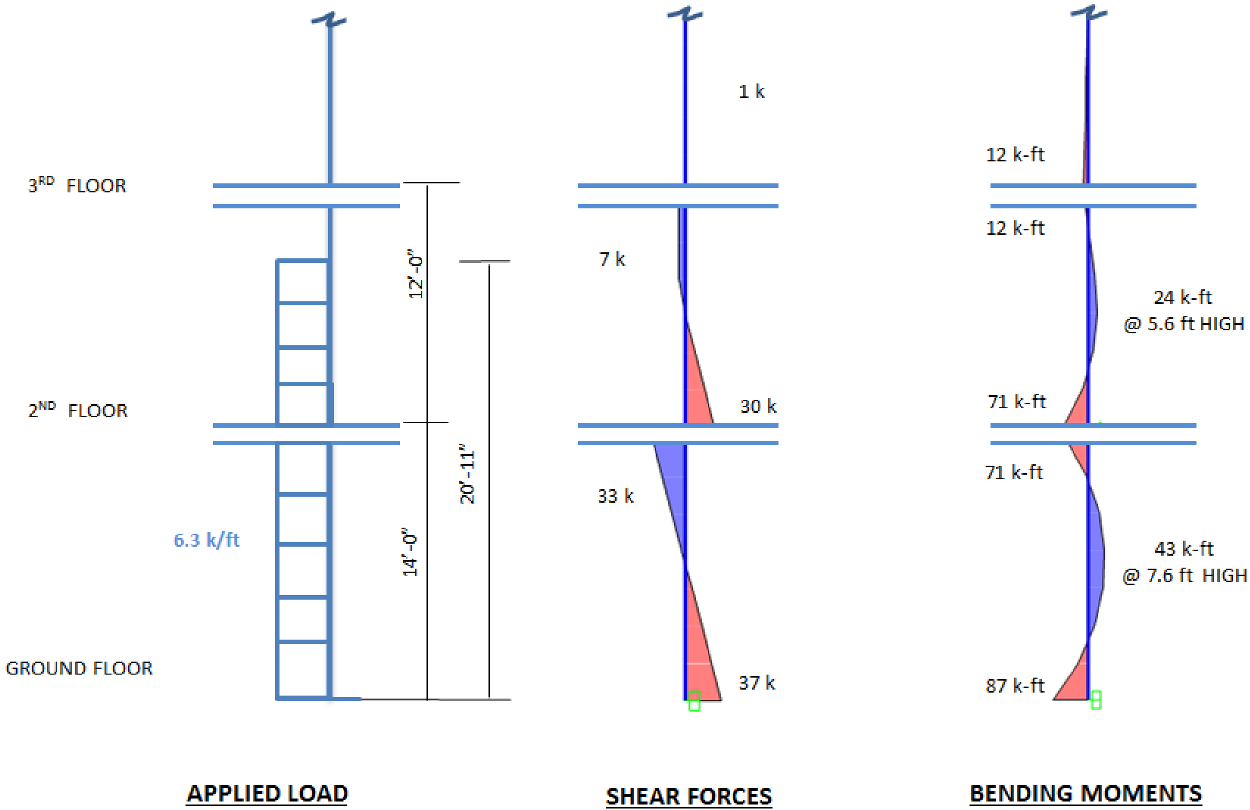

Figure 23: Component hydrodynamic lateral load applied to an interior gravity column, with associated shear force and bending moment diagrams.

Table 5 summarizes the maximum axial load, bending moment and shear forces for all inundated interior gravity columns using the load combinations provided in **section 6.8.3.3** for hydrodynamic drag (Hydro). The cross-section of the existing interior column is shown in **Figure 24**. **Figure 25** shows that the hydrodynamic loads are well within the original column design so no changes are required for the interior columns to satisfy the tsunami design.

Table 5: Results from loading conditions of Hilo office building interior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
		Floor 1		
87	811.8	28	17	1.2D+Ftsu+0.5L (Hydro)
87	566.1	28	17	0.9D+Ftsu (Hydro)
		Floor 2		
71	676.5	20	10	1.2D+Ftsu+0.5L (Hydro)
71	471.75	20	10	0.9D+Ftsu (Hydro)
		Floor 3		
12	541.2	1	1	1.2D+Ftsu+0.5L (Hydro)
12	377.4	1	1	0.9D+Ftsu (Hydro)
		Floor 4		
3	405.9	0	0	1.2D+Ftsu+0.5L (Hydro)
3	283.05	0	0	0.9D+Ftsu (Hydro)
		Floor 5		
1	270.6	0	0	1.2D+Ftsu+0.5L (Hydro)
1	188.7	0	0	0.9D+Ftsu (Hydro)
		Floor 6		
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.35	0	0	0.9D+Ftsu (Hydro)

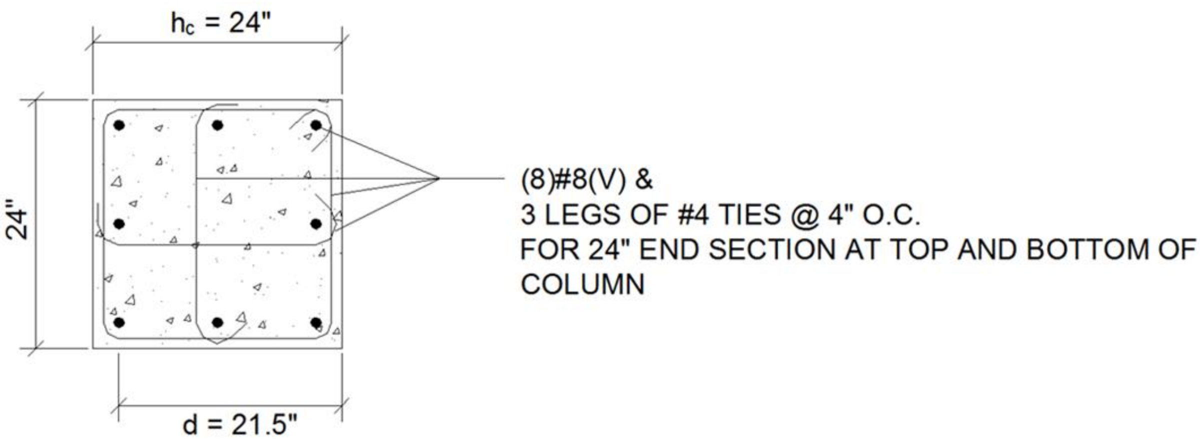

Figure 24: Existing interior gravity load column cross-section at end section (center section has 3-leg #3 ties @5"o.c.).

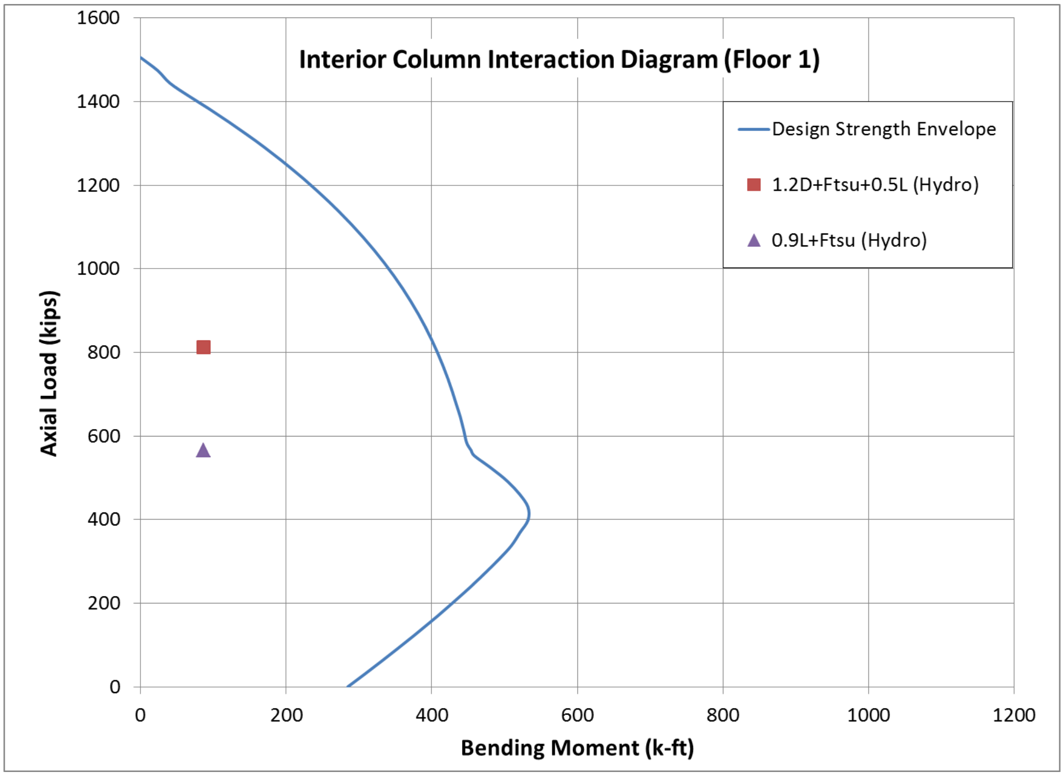

Figure 25: Interaction diagram for existing interior gravity load column showing tsunami hydrodynamic loads

In order to satisfy the ACI 318 shear design, the factored shear force, V_u, must not exceed the design shear strength, ϕV_n, where:

$$\varnothing V_n = \varnothing(V_C + V_S)$$

and $$V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right) bd = 2\sqrt{4000}\left(1 + \frac{811{,}800}{2{,}000\times24\times24}\right) 24 \times 21.5/1{,}000 = 111 \text{ kips.}$$

In the end section, $$V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times21.5}{4\times1{,}000} = 194 \text{ kips}$$

In the center section, $$V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times21.5}{5\times1{,}000} = 85 \text{ kips}$$

Therefore, in the end sections, $\varnothing V_n = 0.75\ (111 + 194) = 229$ k

and in the center sections, $\varnothing V_n = 0.75\ (111 + 85) = 147$ k

At *d:* $V_u = 43$ k < $\phi V_n = 229$ k, therefore the column is adequate for shear in the end section.

At $d + h_c$: $V_u = 26$ k < $\phi V_n = 147$ k, therefore the column is adequate for shear in the center section.

By inspection the remaining interior columns are adequate to resist the tsunami shear force and therefore no changes are required.

Residential Building Tsunami Design

The residential building calculations are similar to the office building presented above. However, the lateral force resisting system consists of four reinforced concrete shear walls while the rest of the column-slab frame supports only gravity loads. The overall building lateral loads from the tsunami will therefore be resisted entirely by the shear walls. In addition, the exterior shear walls facing the inflow and outflow directions must be designed for debris impact. The exterior and interior gravity load columns will be designed for tsunami component loads only, since they do not participate in the lateral force resistance.

Overall Building loading

The overall lateral tsunami loading on the residential building is identical to that for the office building because the overall building width is identical, and the closure coefficient is again controlled by the minimum allowed value of C_{cx} = 0.7. Detailed confirmation of these calculations is provided in Appendix A. **Table 3** provides the flow conditions and resulting hydrodynamic drag force and distributed tsunami loads for Load Cases 1, 2 and 3.

Because the story heights are different for the residential building compared with the office building, the tsunami load tributary to each floor level is different as shown in **Figure 26**.

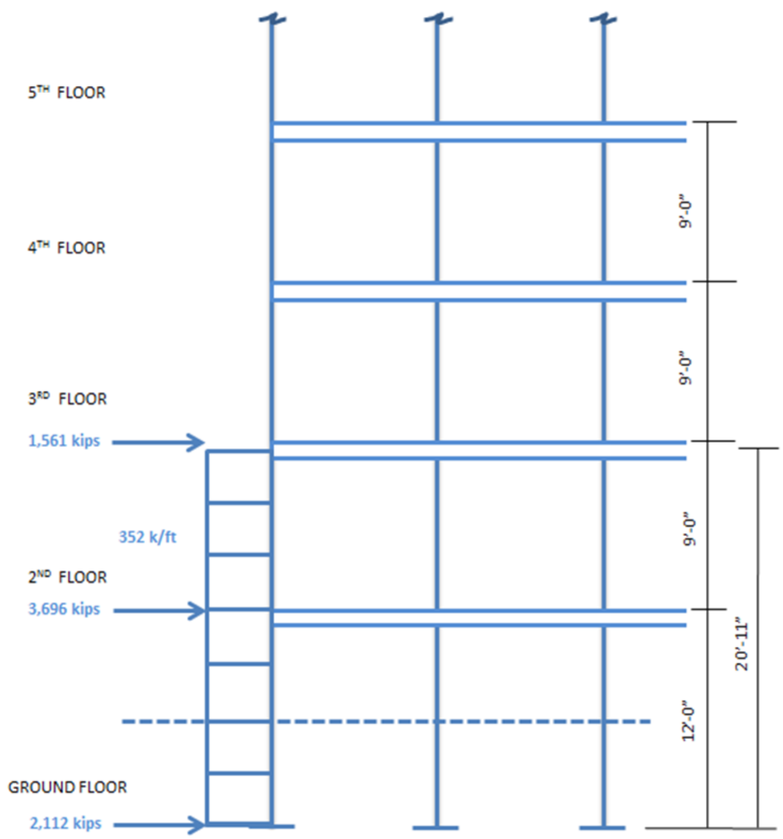

Figure 26: LC2 Tsunami loads on overall Seaside Residential building

The resulting tsunami base shear for this building is therefore V_{TSU} = 1,561 + 3,696 = 5,257 kips. For buildings that are designed for Seismic Design Category D, E or F, ASCE 7-16 allows the design to utilize portion of the seismic overstrength to resist the tsunami loads. Section 6.8.3.4 states that if the tsunami base shear, $V_{TSU} < 0.75\ \Omega_o\ E_h$, then the seismic structural system is adequate to resist the overall building tsunami loads.

From the seismic design of this structure, E_h = 2,435 kips and $\Omega_o = 2.5$, therefore;

$$0.75\Omega_o E_h = 0.75 \times 2.5 \times 2{,}435 = 4{,}566\ kips\ < V_{TSU} = 5{,}257\ kips$$

Therefore the lateral force resisting system is not adequate to resist the tsunami loads and must be strengthened. ETABS was used to repeat the seismic design for a base shear of:

$$E_h = \frac{V_{TSU}}{0.75\Omega_o} = \frac{5{,}257}{0.75\times 2.5} = 2{,}804\ kips$$ in place of the original E_h = 2,435 kips.

The ETABS model for the Seaside location is shown in **Figure 27**. The resulting axial loads, shear forces and bending moments at the base of each shear wall are shown in

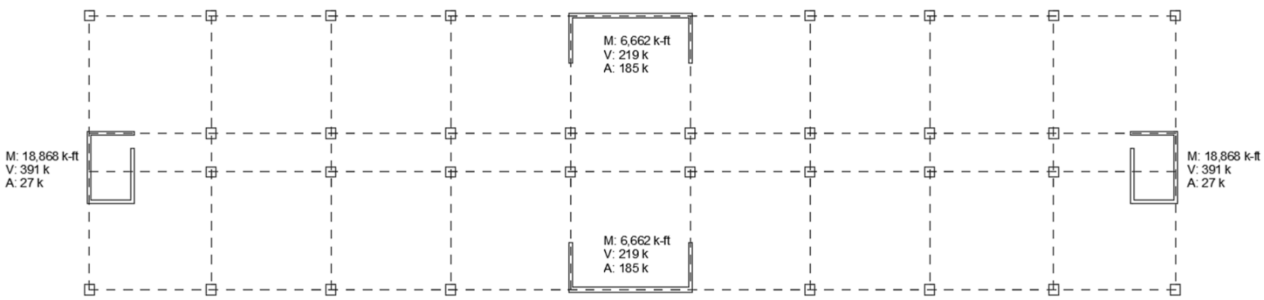

Figure 28.

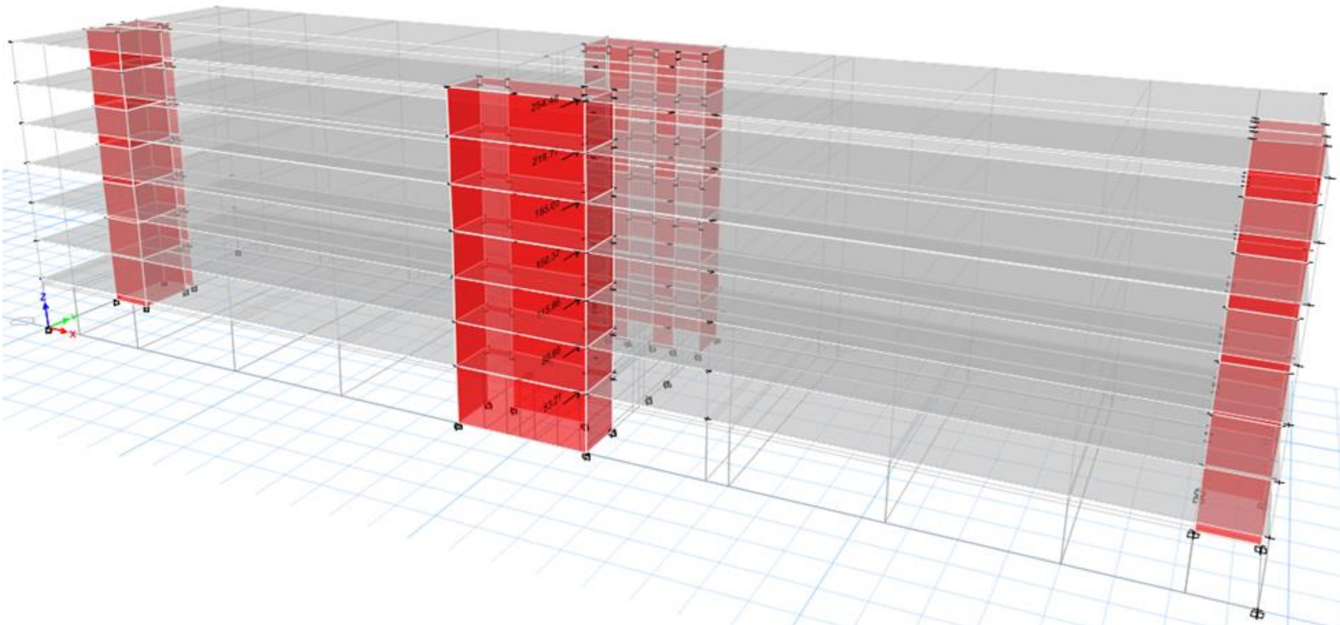

Figure 27: ETABS computer model of residential building subjected to elevated seismic loads to meet the tsunami demand at the Seaside location

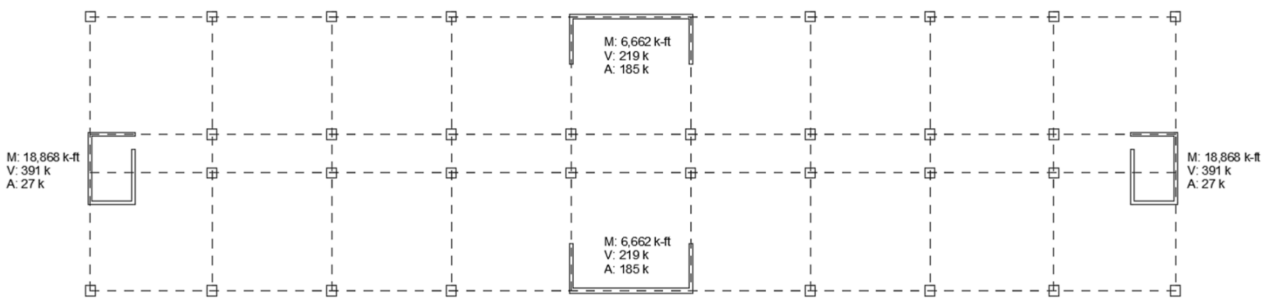

Figure 28: ETABS output of axial load, shear force and bending moment at the base of each structural wall for the Seaside location

Table 6 summarizes the maximum axial load, bending moment and shear forces for all inundated shear walls using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and long impact (Impact). The original shear wall designs will now be evaluated for these load combinations and modified if necessary.

Table 6: Overall residential building shear wall load results for Seaside location (Floor 1 - 2)

Moment k-ft	Axial Load Kips	Shear Kips	Load Combination
Floor 1			
Earthquake			
5,753	630	161	1.2D+Ftsu+0.5L (Elv/Mech)
5,753	399	161	0.9D+Ftsu (Elv/Mech)
-5,753	630	161	1.2D+Ftsu+0.5L (Elv/Mech)
-5,753	399	161	0.9D+Ftsu (Elv/Mech)
16,390	796	340	1.2D+Ftsu+0.5L (Stairs)
16,390	566	340	0.9D+Ftsu (Stairs)
-16,390	796	340	1.2D+Ftsu+0.5L (Stairs)
-16,390	566	340	0.9D+Ftsu (Stairs)
Tsunami			
6,623	601	185	1.2D+Ftsu+0.5L (Elv/Mech)
6,623	371	185	0.9D+Ftsu (Elv/Mech)
-6,623	601	185	1.2D+Ftsu+0.5L (Elv/Mech)
-6,623	371	185	0.9D+Ftsu (Elv/Mech)
18,868	793	391	1.2D+Ftsu+0.5L (Stairs)
18,868	563	391	0.9D+Ftsu (Stairs)
-18,868	793	391	1.2D+Ftsu+0.5L (Stairs)
-18,868	563	391	0.9D+Ftsu (Stairs)
Floor 2			
Earthquake			
3,930	539	138	1.2D+Ftsu+0.5L (Elv/Mech)
3,930	342	138	0.9D+Ftsu (Elv/Mech)
-3,930	539	138	1.2D+Ftsu+0.5L (Elv/Mech)
-3,930	342	138	0.9D+Ftsu (Elv/Mech)
12,310	686	337	1.2D+Ftsu+0.5L (Stairs)
12,310	489	337	0.9D+Ftsu (Stairs)
-12,310	686	337	1.2D+Ftsu+0.5L (Stairs)
-12,310	489	337	0.9D+Ftsu (Stairs)
Tsunami			
4,525	515	159	1.2D+Ftsu+0.5L (Elv/Mech)
4,525	317	159	0.9D+Ftsu (Elv/Mech)
-4,525	515	159	1.2D+Ftsu+0.5L (Elv/Mech)
-4,525	317	159	0.9D+Ftsu (Elv/Mech)
14,171	684	388	1.2D+Ftsu+0.5L (Stairs)
14,171	486	388	0.9D+Ftsu (Stairs)
-14,171	684	388	1.2D+Ftsu+0.5L (Stairs)
-14,171	486	388	0.9D+Ftsu (Stairs)

The original shear wall reinforcement layout is shown in **Figure 29** through **Figure 34** for the first three floor levels of the elevator/mechanical room shear walls and the stairwell shear walls at the Seaside location.

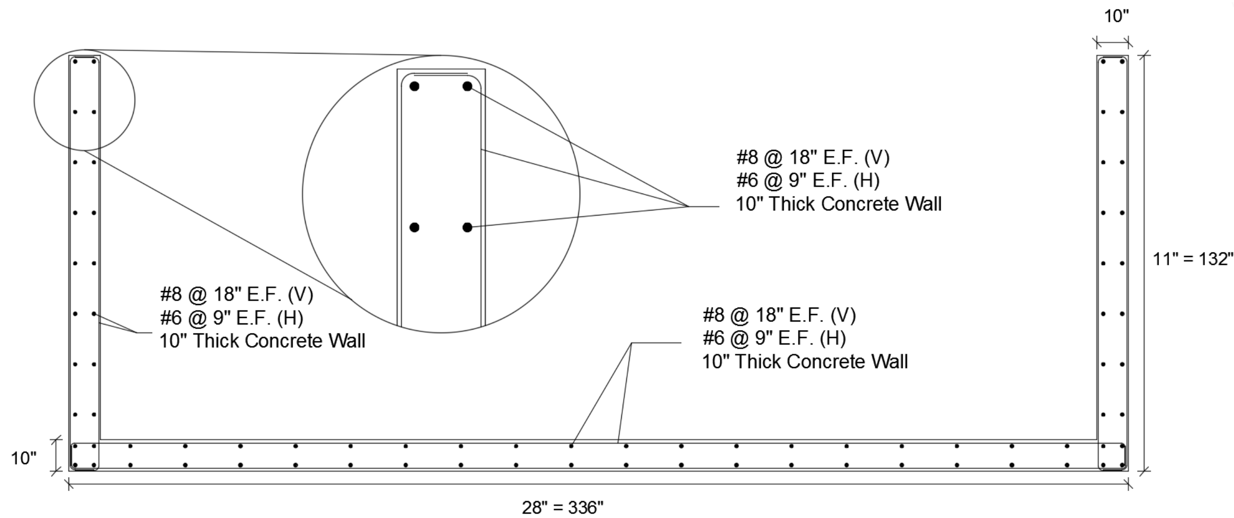

Figure 29: Original Elevator/ Mech. Room shear wall cross-section at the ground floor level

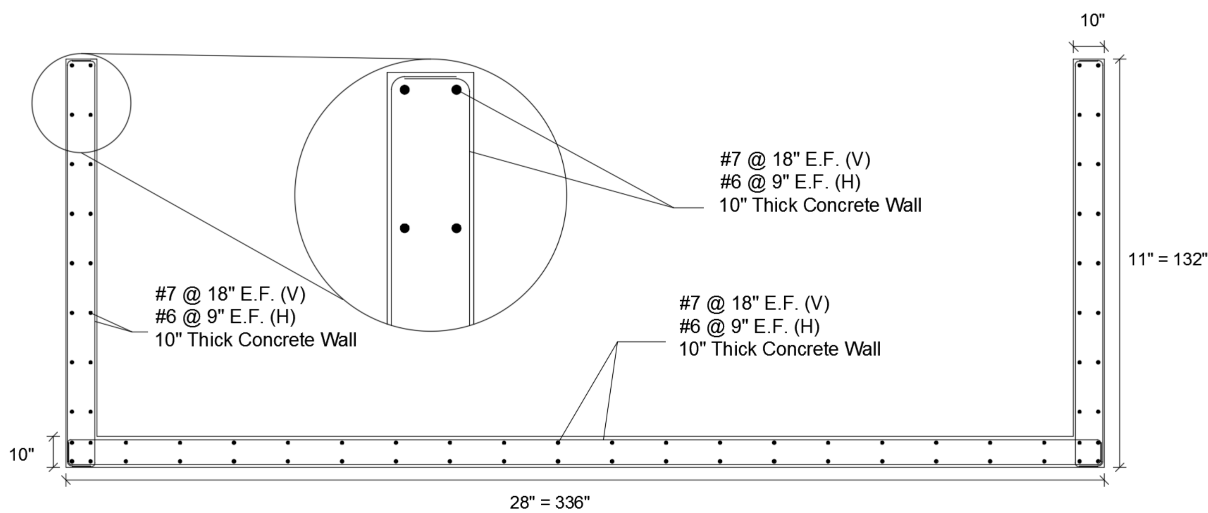

Figure 30: Original Elevator/ Mech. Room shear wall cross-section at the 2nd floor level

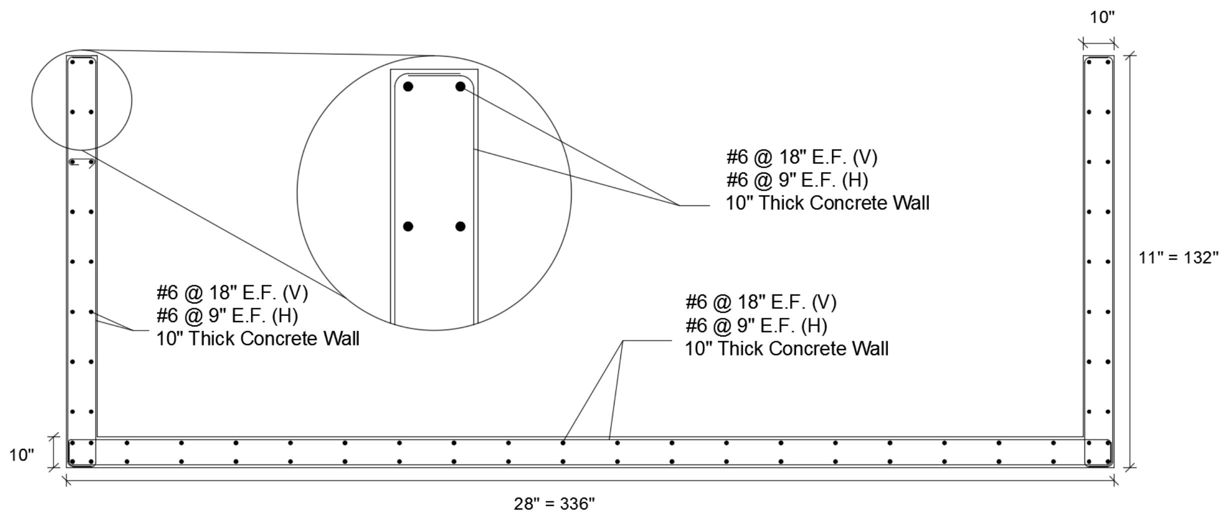

Figure 31: Original Elevator/ Mech. Room shear wall cross-section at the 3rd floor level

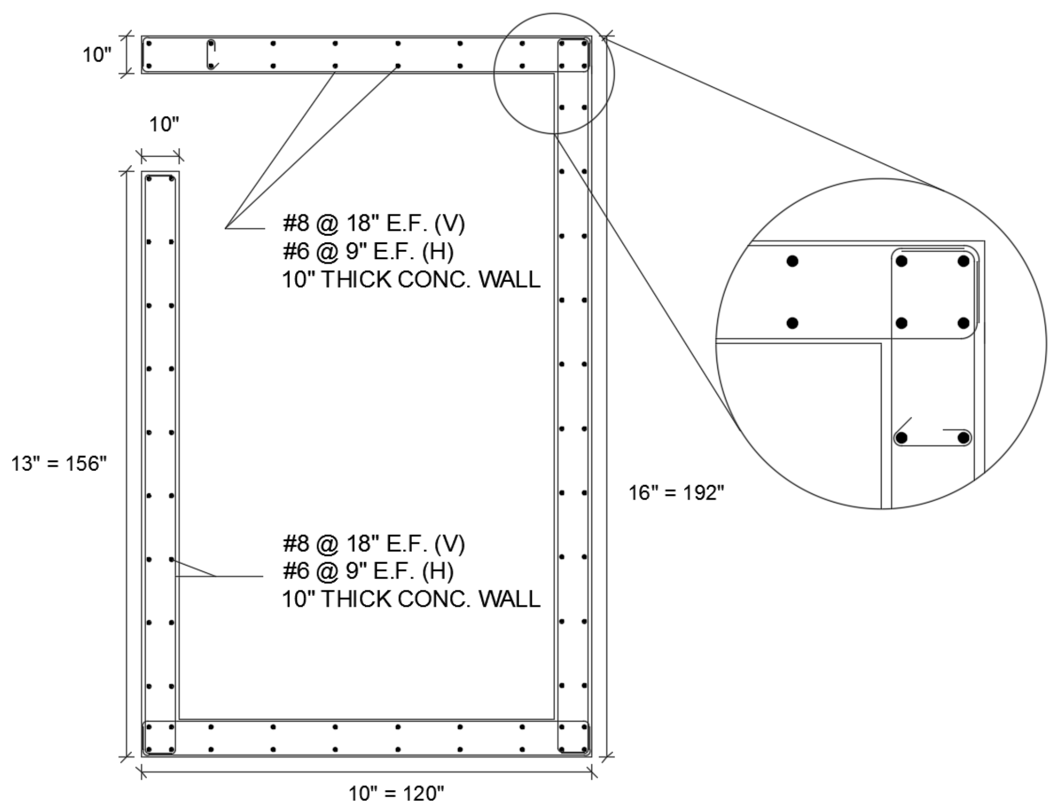

Figure 32: Original stairwell shear wall cross-section at the ground floor level

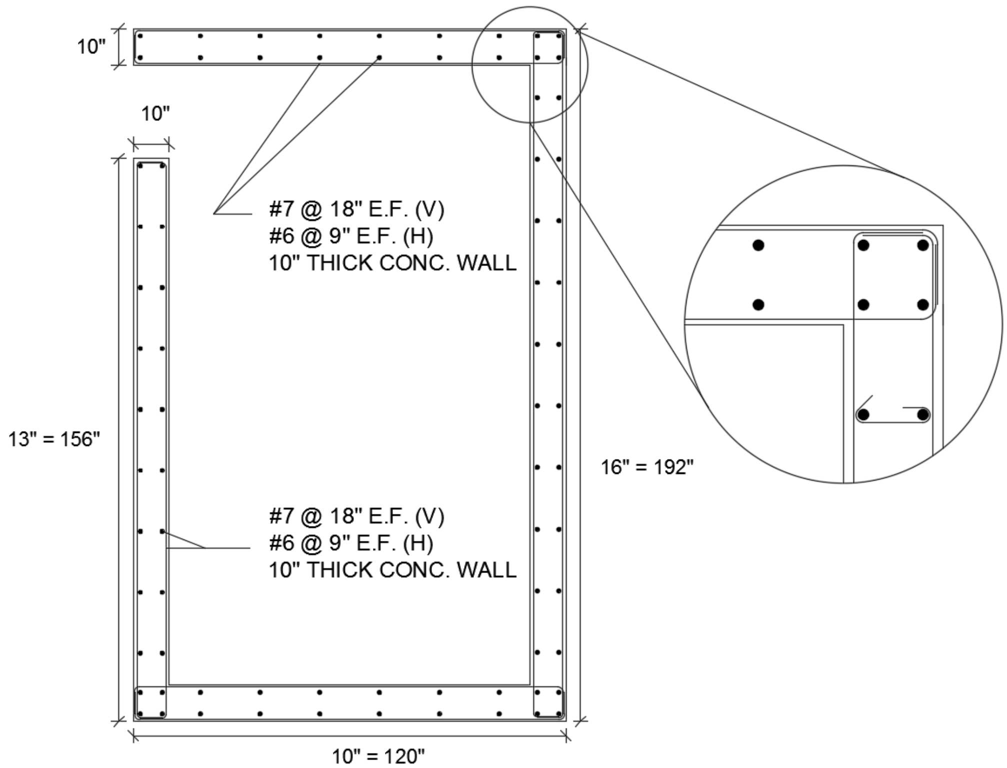

Figure 33: Original stairwell shear wall cross-section at the 2nd floor level

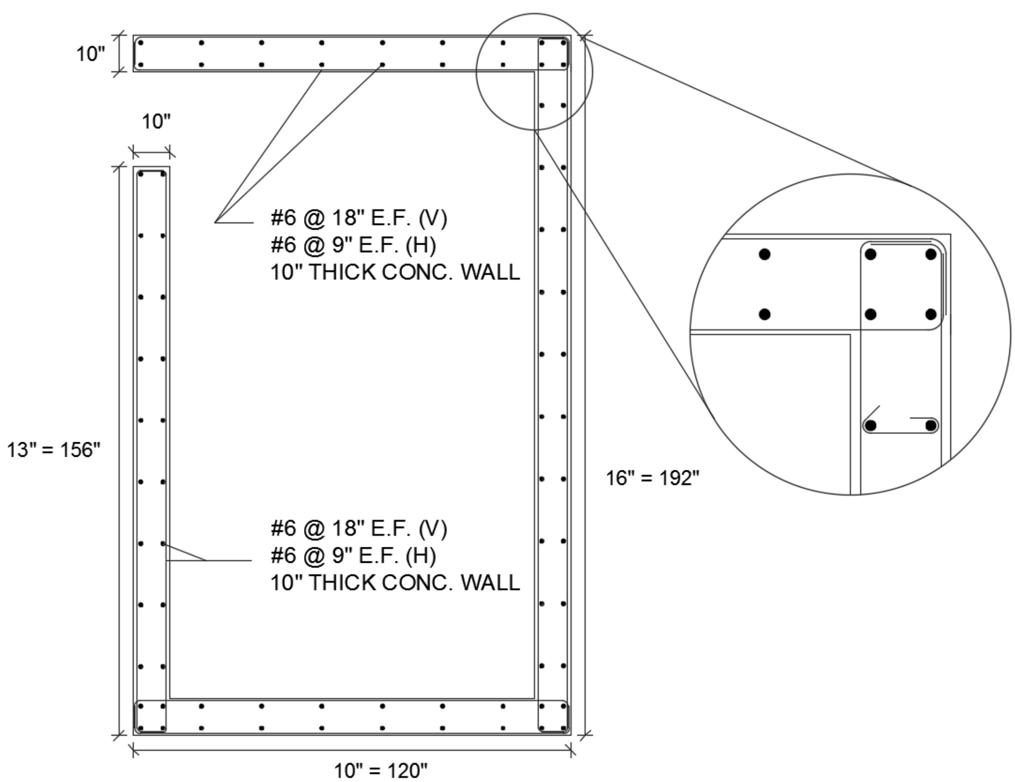

Figure 34: Original stairwell shear wall cross-section at the 3rd floor level

Existing Exterior Shear Wall Design for Combined Gravity and Tsunami Loads

The elevator/mechanical room and stairwell shear walls will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**.

Figure 35 through **Figure 38** show the interaction diagrams for the first two floors of the shear walls at the elevator/mechanical room and stairwells for the tsunami load combinations at the Seaside location. The original wall reinforcement is adequate to resist the slightly larger seismic loads required to satisfy the tsunami design.

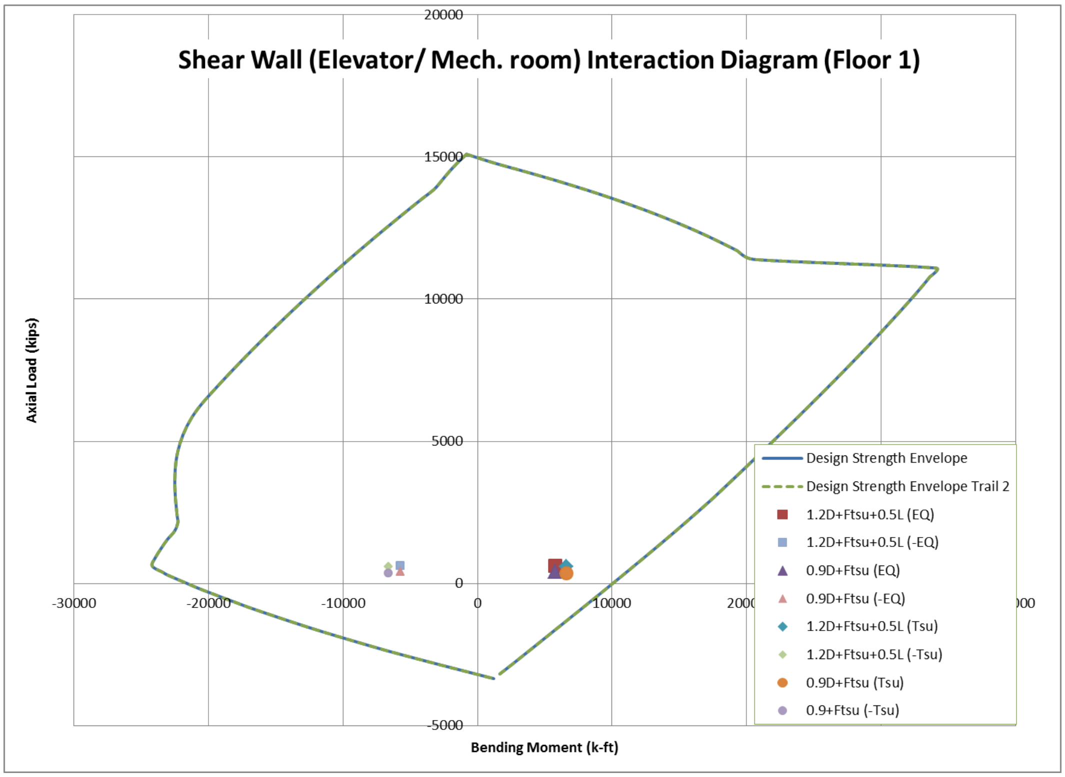

Figure 35: Interaction diagram for elevator/mechanical room shear walls at the ground floor level at the Seaside location

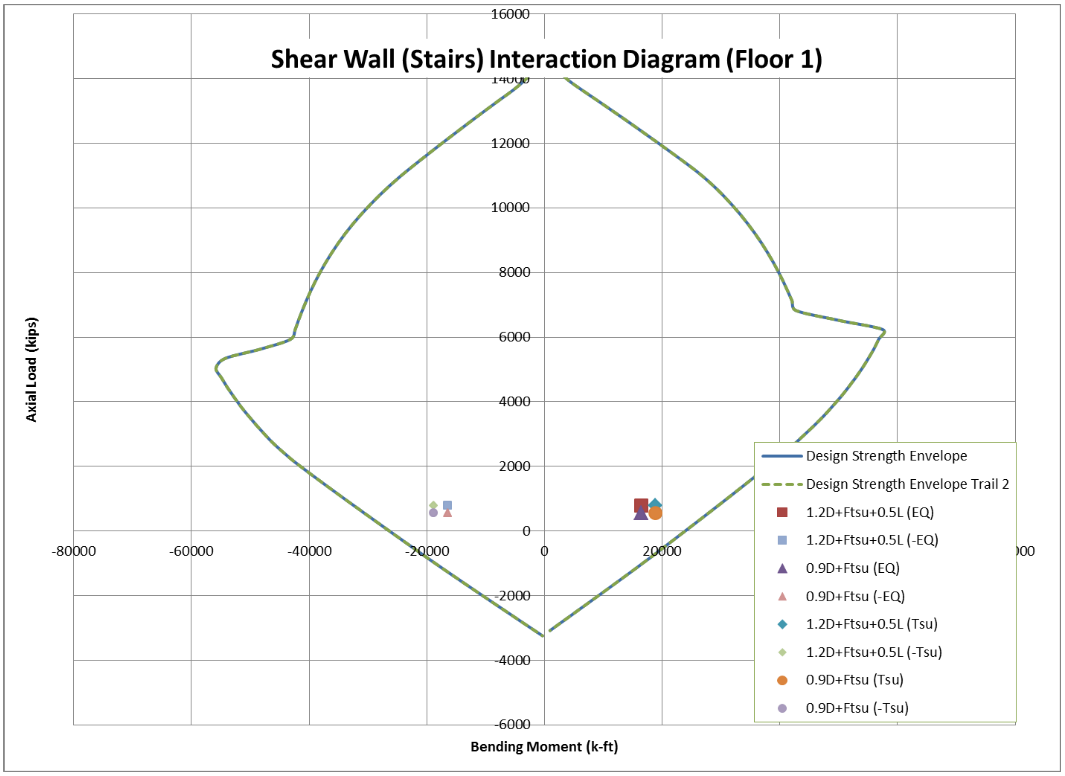

Figure 36: Interaction diagram for stairwell shear walls at the ground floor level at the Seaside location

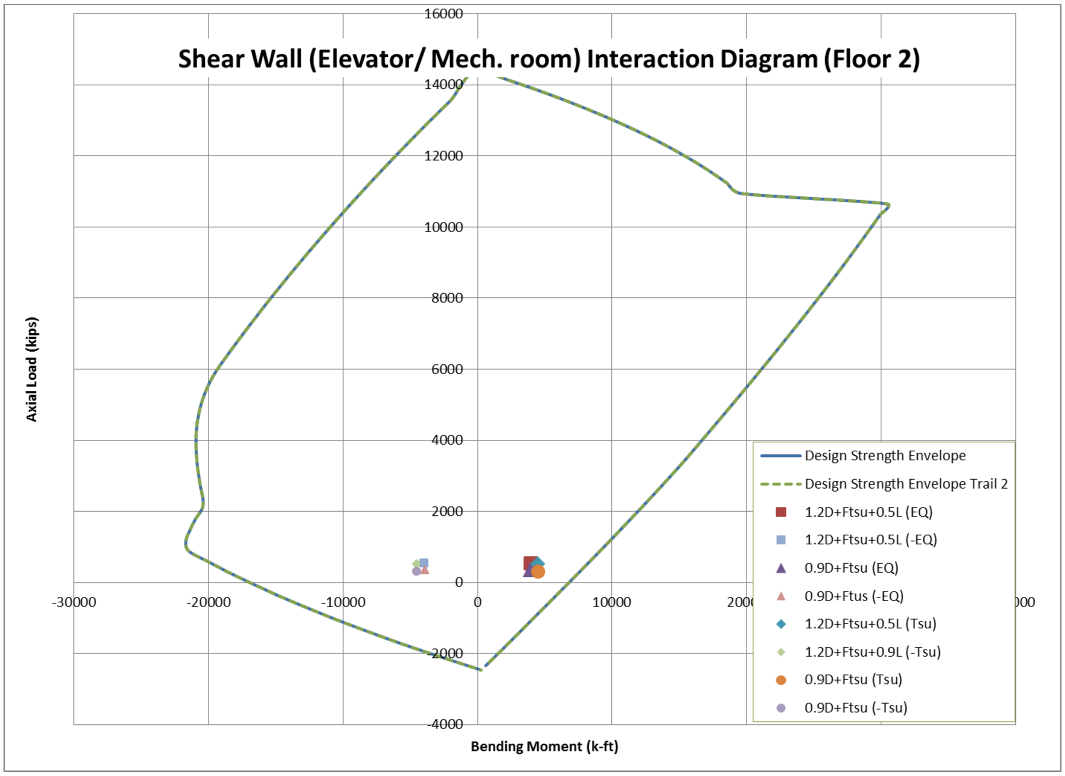

Figure 37: Interaction diagram for elevator/mechanical room shear walls at the second floor level at the Seaside location

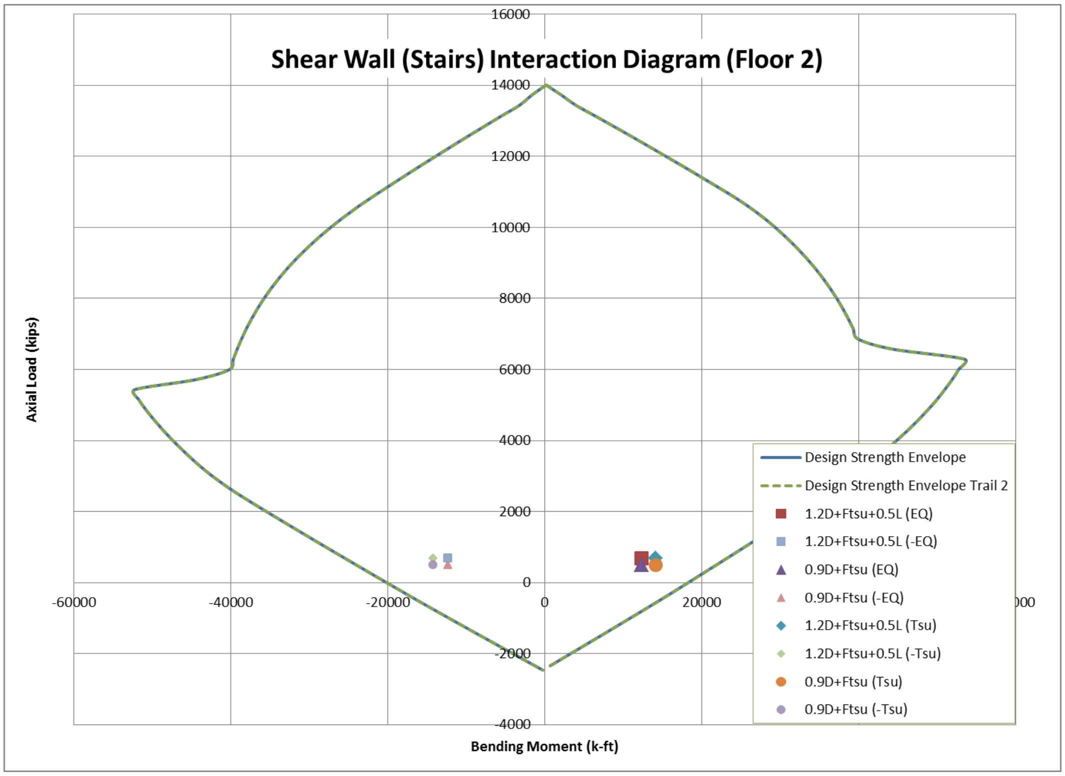

Figure 38: Interaction diagram for stairwell shear walls at the second floor level at the Seaside location

Design of Shear Walls for Overall Building Shear Loads

Shear capacity of existing elevator/mechanical room shear walls:

At the first floor level, the two flanges of the C-shaped shear walls must resist the tsunami base shear (V_{TSU} = 185 kips, see **Table 6**) due to the critical axial load combination per **Eqn. 6.8.3.3.-1b** (0.9D + F_{TSU}).

Shear capacity of existing shear wall (Elevator/mechanical room):

$\emptyset V_n = \emptyset(V_C + V_S)$

Where $V_C = 2\,\lambda\,\sqrt{f'_c}\,b\,d = 2 \times 1\,\sqrt{4000}\ \times 10 \times 105.6/1{,}000$ = 133.6 kips for each flange.

$d = 0.8 \times L_w = 0.8 \times 132" = 105.6\ in$

L_w = 11'= 132 in (Length of flange resisting shear)

b = 10" (Wall thickness)

$\emptyset$ = 0.75

$\emptyset V_c$ = 0.75 (2 x 133.6) = 200 kips > V_{tsu} = 185 kips therefore the shear capacity of the original wall is adequate for the tsunami loads.

At the first floor level, the stairway shear walls must resist the tsunami base shear (V_{TSU} = 388 kips, see **Table 6**) due to the critical axial load combination per **Eqn. 6.8.3.3.-1b** (0.9D + F_{TSU}).

Shear Capacity of existing stairway shear walls:

$ØV_n = Ø(V_C + V_S)$

where $V_c = V_{c1} + V_{c2}$ is the concrete shear capacity of the two walls parallel to the loading direction.

$V_{C1} = 2\,\lambda\sqrt{f'_c}\,b\,d = 2 \times 1\sqrt{4000} \times 10 \times 154/1{,}000$ = 194 kips for the full length wall,

and $V_{C2} = 2\,\lambda\sqrt{f'_c}\,b\,d = 2 \times 1\sqrt{4000} \times 10 \times 125/1{,}000$ = 158 kips for the wall with doorway.

So, $V_C = V_{C1} + V_{C2} = 194 + 158 = 352$ kips

where $d_1 = 0.8 \times L_{w1} = 0.8 \times 192" = 154\ in$

$d_2 = 0.8 \times L_{w2} = 0.8 \times 156" = 125\ in$

L_{w1} = 16'= 192 in

L_{w2} = 13'= 156 in

b = 10" (wall thickness)

and $Ø = 0.75$.

The two walls parallel to the flow direction are reinforced with #6@9" o.c. in each face of the walls. This provides the following shear capacity:

$V_{S1} = \frac{A_t f_y d}{s} = \frac{(2\times0.44)\times60{,}000\times154}{9}$ = 901 kips

$V_{S2} = \frac{A_t f_y d}{s} = \frac{(2\times0.44)\times60{,}000\times125}{9}$ = 732 kips

$V_s = V_{s1} + V_{s2} = 901 + 732 = 1{,}633$ kips

A_t = 0.44 in (#6 Rebar)

S = 9 in (Spacing).

Therefore, $ØV_n = Ø(V_C + V_S) = 0.75\ (352+1{,}633) = 1{,}489$ kips > V_{TSU} = 388 kips, therefore the stairwell shear walls are adequate for shear.

By similar analysis, it was determined that the existing shear walls at the remaining floors are adequate to resist the tsunami shear forces.

Hydrodynamic Component Loads

Shear Walls

In addition to resisting the overall building tsunami lateral loads, the shear walls must also be designed for component loads due to hydrodynamic and debris loading. Shear walls located on the exterior of the building, such as the elevator shafts and mechanical rooms in the residential building, must be designed for hydrodynamic drag including debris damming, and for debris impact. Shear walls located in the building interior, or on the ends of the building that are parallel to the tsunami flow direction, such as the stairwell shear walls in the residential building, only need to be designed for hydrodynamic drag without consideration of debris damming or debris impact.

Since tsunami bores are anticipated at this location, the lateral load on the structural walls is given by **Eqn. 6.10-5a** or **Eqn. 6.10-5b**, depending on the flow depth relative to the wall width:

Eqn. 6.10-5a: $F_W = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$

Eqn. 6.10-5b: $F_W = \frac{3}{4}\rho_s I_{tsu} C_d b(h_e u^2)$ when $\frac{b_w}{h_e} \geq 3$

Where C_d = 2.0 for a wall per **Table 6.10-2.**

Elevator Walls: For the elevator and machine room walls, b = 28 ft.

For Load Case 2, where h_e = 20.93 ft and u = 37.92 fps, $\frac{b_w}{h_e} = \frac{28'}{20.93} = 1.34 \ngeq 3 \therefore \text{Eqn. 6.10-5a applies.}$

Therefore, for the 28' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 28(20.93 \times 37.92)/1000 = 1{,}854 kips$

These loads are applied to the walls as a uniformly distributed pressure of 1,854/(28x20.93) = 3,163 psf over the lower 20.93 ft of the walls. For comparison with the debris impact loads (see next section), this pressure is applied to a 5.67 ft. wide vertical strip of wall, resulting in a uniformly distributed lateral load of 5.67x3163 = 18 kip/ft, as shown in **Figure 40**.

It is possible that the inundation occurs as a series of bores each with height less than 2/3h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} = \frac{1}{3} \rightarrow h_e = \frac{b_w}{3'} = \frac{28'}{3'}$ = 9.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 9.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure 39**. The normalized flow depth is h/h_{max} = 9.33'/31.4' = 0.297. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.08. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.78. Therefore the flow velocity is u = 0.78 x 37.92 = 29.6 fps. The bore loading is therefore computed for h_e = 9.33 ft and u = 29.6 fps.

For the 28' wide elevator wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 28(9.33 \times 29.6^2)/1000 = 755\ kips$

This load is applied to the wall as a uniformly distributed pressure of 755/(28x9.33) = 2,890 psf over the lower 9.33 ft of the wall. This will not govern when compared with the pressure from hydrodynamic drag using **Eqn. 6.10-5a**.

Stairwell Walls: For the stairwall walls, the exposed width is *b* = 10 ft.

For Load Case 2, where h_e = 20.93 ft and u = 37.92 fps, $\frac{b_w}{h_e} = \frac{10'}{20.93'} = 0.48 \ngeq 3 \therefore \text{Eqn. 6.10-5a applies.}$

Therefore, for the 10' wide wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 10(20.93 \times 37.92^2)/1000 = 662\ kips$

This load is applied to the wall as a uniformly distributed pressure of 662/(10x20.93) = 3,163 psf over the lower 20.93 ft of the wall.

It is possible that the inundation occurs as a series of bores each with height less than 2/3h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} = \frac{1}{3} \rightarrow h_e = \frac{b_w}{3'} = \frac{10'}{3'}$ = 3.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 3.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure 39**. The normalized flow depth is h/h_{max} = 3.33'/31.4' = 0.106. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.03. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.45. Therefore the flow velocity is u = 0.45 x 37.92 = 17.1 fps. The bore loading is computed for h_e = 3.33 ft and u = 17.1 fps.

Therefore for the 10' wide wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 10(3.33 \times 17.1^2)/1000 = 32.1\ kips.$

This load is applied to the wall as a uniformly distributed pressure of 32.1/(10x3.33) = 963 psf over the lower 3.33 ft of the wall. This will not govern when compared with the pressure from hydrodynamic drag using Eqn. **6.10-5a**.

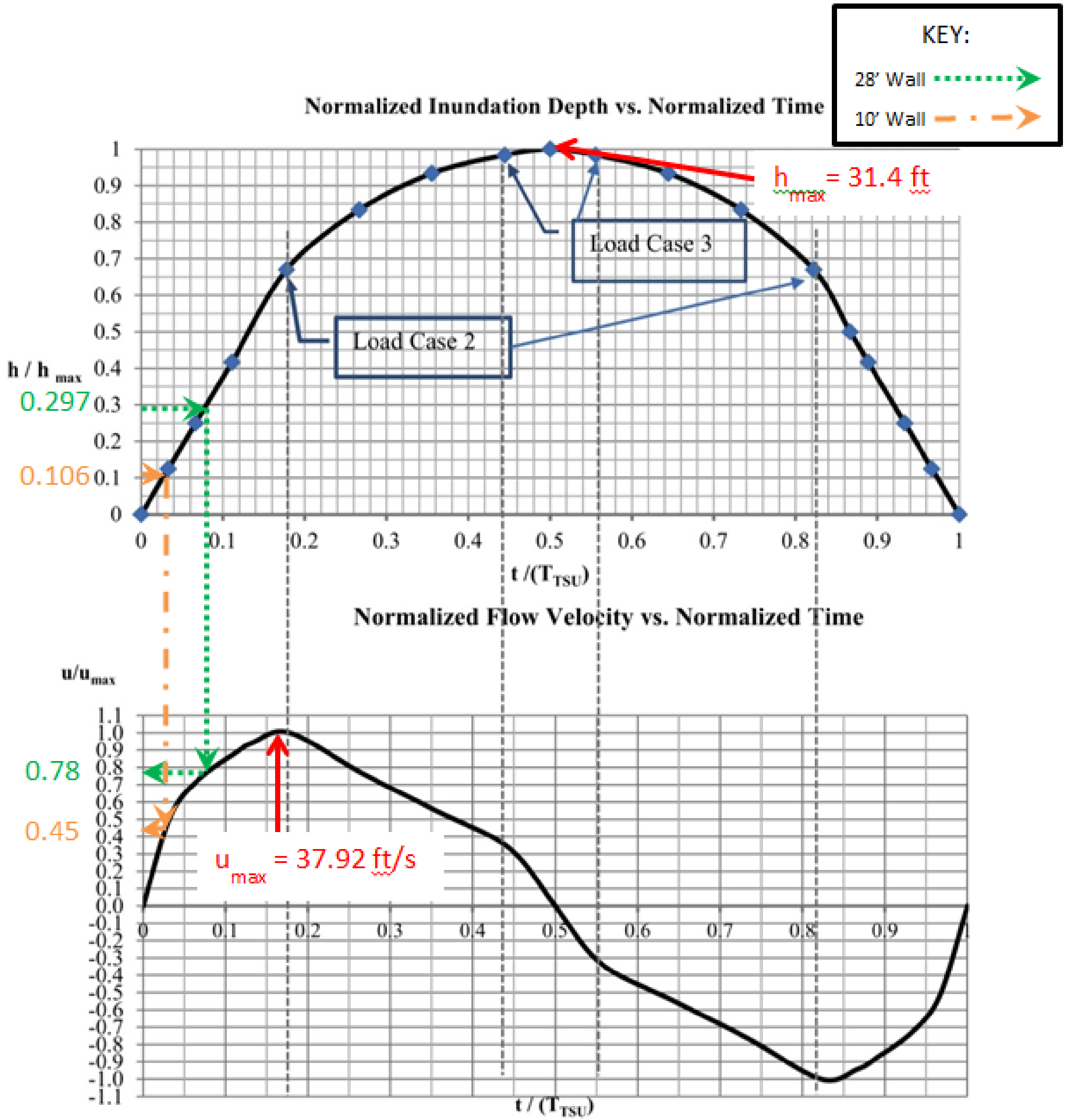

Figure 39: Flow velocity determination for various flow depths.

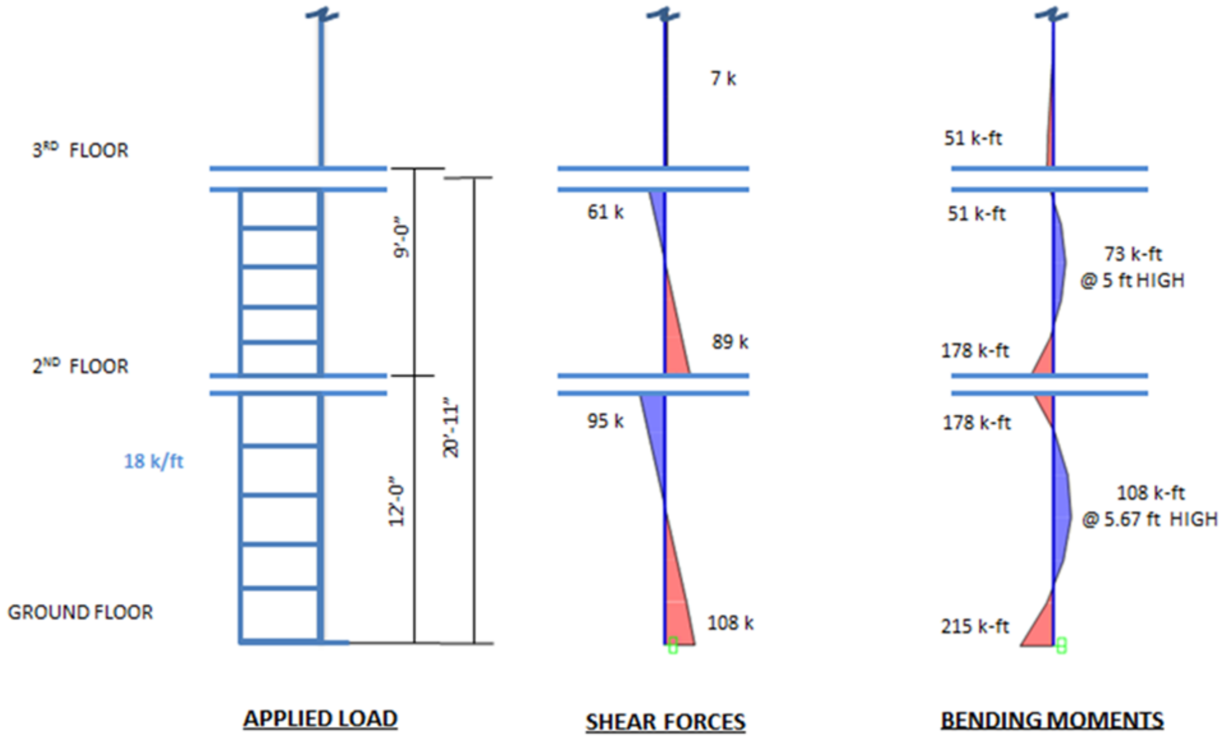

Figure 40: Hydrodynamic loading on 5.67 foot wide section of exterior wall of Seaside residential building due to Load Case 2

Debris Impact Component Loads

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is resisted by an effective wall width of $b_e = h/2$, where h is the clear story height. Therefore b_e = (12-8/12)/2 = 5.67 ft as shown in **Figure 41**. The debris impact load must be applied to all submerged sections of the shear wall, at all locations that could produce maximum bending moment and/or maximum shear in the wall. The maximum shear force occurs when the load is applied at a point just above or just below the slab at each inundated floor. The maximum bending moment occurs when the load is applied at the mid-height of the clear wall height. The resulting shear force and bending moment diagrams for log impact at a distance "*d*" from the top end of the ground floor wall are shown in **Figure 42**, where *d* is the wall effective depth.

Shear Wall Component Design

A 5.67 ft. width of wall is analyzed as a column element subjected to either the hydrodynamic or debris impact loads determined above. The wall must also support the dead load due to gravity loads.

Gravity Load Calculation

For the 28 ft. wide elevator and mechanical room shear walls, the floor slab gravity load tributary width is 5.5 ft. Gravity loads will be computed per foot width of wall.

The Dead Load at the base of the wall is:

$$P_D = [128(5.5)(1)(6)+123(5.5)(1)+0.83(1)(150)(66)]/1000 = 13.1 \text{ k/ft}$$

Floor Live load reduction: Reduction Factor = $0.25+15/[1(5.5)(28)(6)]^{0.5} = 0.743$

$$P_L = 0.743[55(5.5)(1)(6)]/1000 = 1.35 \text{ k/ft}$$

Roof Live Load reduction: Reduction Factor = R_1R_2= [1.2-(0.001)(5.5)(28)](1.0) = 1.05, Use 1.0

$$P_{Lr} = 20(5.5)(1)/1000 = 0.110 \text{ k/ft}$$

These per foot values are multiplied by 5.67 ft. to get the axial loads on the shear wall component being designed.

Table 7 summarizes the maximum axial load, bending moment and shear forces for all inundated exterior shear walls (for a 5.67 ft wall width) using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and log impact (Impact). The original wall designs are then evaluated for these load combinations and modified if necessary. **Figure 43** shows that the ground floor elevator wall reinforcement was inadequate to resist either the hydrodynamic or debris impact loading conditions. Therefore the reinforcement was increased to produce an interaction diagram that exceeded the applied loads.

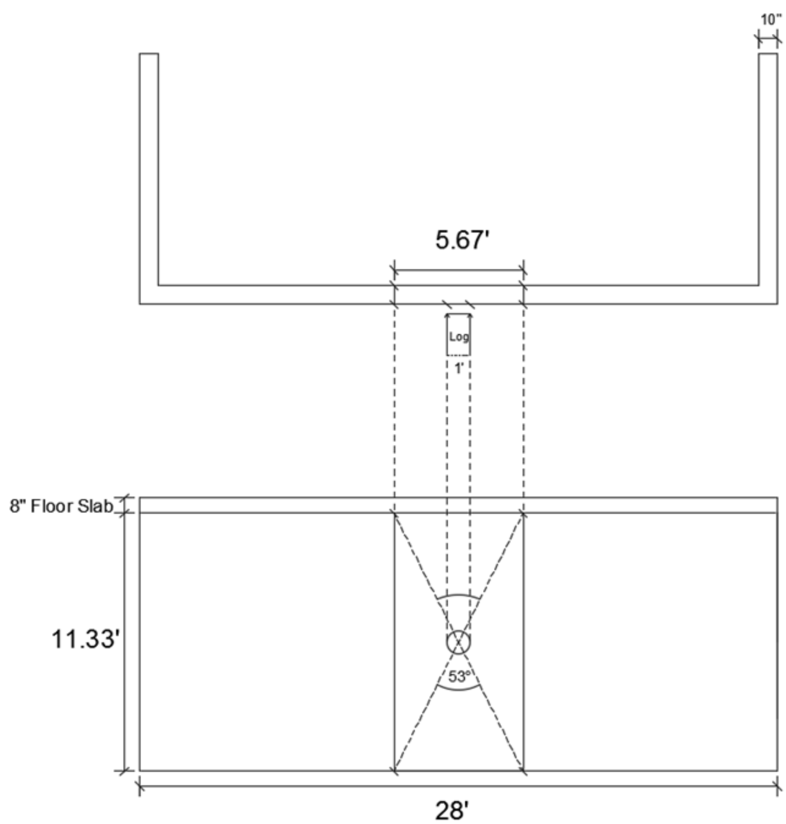

Figure 41: Log debris impact on wall showing effective width of 5.67 ft.

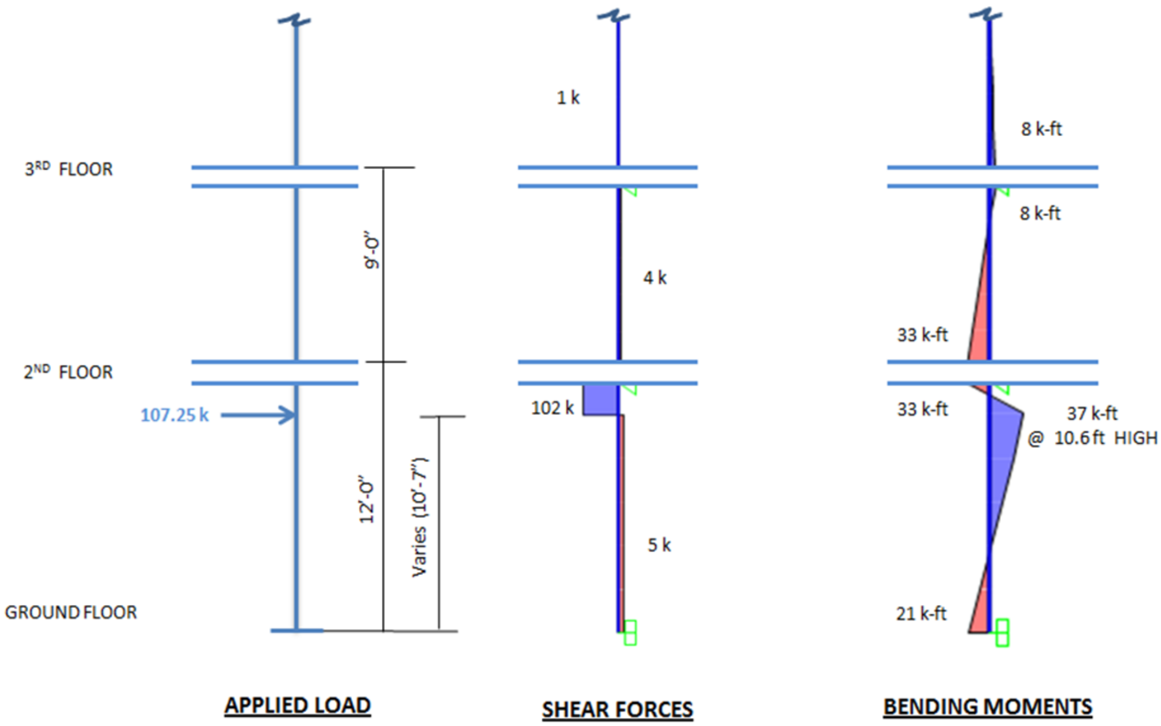

Figure 42: Impact load applied at *d* away from the end of column on the ground floor

Table 7: Results from loading conditions of Seaside residential building exterior shear wall (for 5.67ft width)

Moment K-ft	Axial Load Kips	Shear @ d Kips	Load combination
Floor 1			
215	92.91	95	1.2D+Ftsu+0.5L (Hydro)
215	6.89	95	0.9D+Ftsu (Hydro)
176	92.91	102	1.2D+Ftsu+0.5L (Impact)
176	6.89	102	0.9D+Ftsu (Impact)
Floor 2			
178	79.63	76	1.2D+Ftsu+0.5L (Hydro)
178	5.90	76	0.9D+Ftsu (Hydro)
168	79.63	101	1.2D+Ftsu+0.5L (Impact)
168	5.90	101	0.9D+Ftsu (Impact)
Floor 3			
51	66.36	7	1.2D+Ftsu+0.5L (Hydro)
51	4.92	7	0.9D+Ftsu (Hydro)
167	66.36	101	1.2D+Ftsu+0.5L (Impact)
167	4.92	101	0.9D+Ftsu (Impact)
Floor 4			
12	53.09	2	1.2D+Ftsu+0.5L (Hydro)
12	3.93	2	0.9D+Ftsu (Hydro)
80	53.09	100	1.2D+Ftsu+0.5L (Impact)
80	3.93	100	0.9D+Ftsu (Impact)
Floor 5			
3	39.82	0	1.2D+Ftsu+0.5L (Hydro)
3	2.95	0	0.9D+Ftsu (Hydro)
25	39.82	2	1.2D+Ftsu+0.5L (Impact)
25	2.95	2	0.9D+Ftsu (Impact)
Floor 6			
1	26.54	0	1.2D+Ftsu+0.5L (Hydro)
1	1.97	0	0.9D+Ftsu (Hydro)
6	26.54	0	1.2D+Ftsu+0.5L (Impact)
6	1.97	0	0.9D+Ftsu (Impact)
Floor 7			
0	13.27	0	1.2D+Ftsu+0.5L (Hydro)
0	0.98	0	0.9D+Ftsu (Hydro)
1	13.27	0	1.2D+Ftsu+0.5L (Impact)
1	0.98	0	0.9D+Ftsu (Impact)

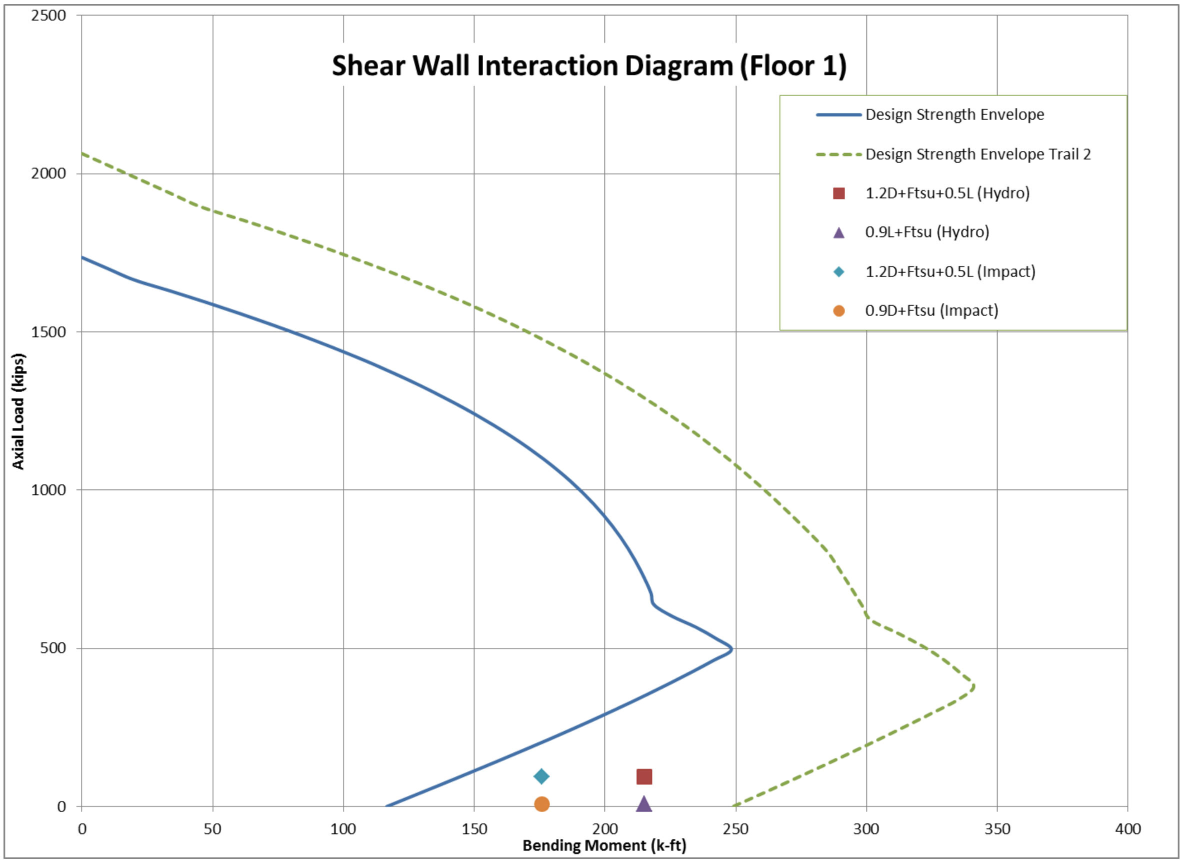

Figure 43: Interaction diagram for typical ground floor exterior wall segment showing tsunami load combinations

The shear capacity of the walls must also be checked and augmented if necessary. A log impact could result in a shear failure due to two-way "punching" shear at any location on the wall, or due to one-way "beam" shear at the top and bottom of the wall.

Punching shear due to a log impact is accounted for using the ACI-318 punching shear equation for concrete resistance given as:

$$ØV_n = Ø4\sqrt{f'_c}\, b_o d$$

where: ϕ = strength reduction factor = 0.75 for shear

f_c' = concrete strength = 4,000 psi

d = Wall thickness – cover – bar radius = 10"-0.75"- ½" = 8.75"

and, $b_o = \pi*(d + \text{Log Dia.}) = \pi*(8.75" + 12") = 65.2"$

Therefore, $ØV_n = Ø4\sqrt{f'_c}\, b_o d = 0.75*4*\sqrt{4000}*65.2*8.75 = 108 \text{ k} > 107.25 \text{ k}$

So the wall will be adequate for punching shear without any shear reinforcement.

One-way or "beam" shear is affected by the axial load on the wall. The critical load combination given by **Eqn. 6.8.3.3.-1b** uses a reduced axial load as (0.9*D* + F_{TSU}), resulting in P_u = 6.89 kips (**Table 7**).

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\, b\, d = 2\left(1 + \frac{807}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.75/1{,}000$ = 75 kips

$N_u = \left(\frac{P_u \times 1{,}000}{l_w}\right) \times b = \left(\frac{6.89 \times 1{,}000}{580}\right) \times 68 = 807\ lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

Therefore, $ØV_n = Ø(V_C + V_S)$ = 0.75 (75+0) = 56 kips

But V_{tsu} = 102 kips > $ØV_n$ = 56 kips, therefore shear reinforcement is required to provide 44 kips shear capacity.

Shear studs will be used in place of traditional stirrups as they are more effective in thin elements such as slabs and walls. The following calculations determine the shear stud arrangement required.

Shear Capacity for New Shear Wall (10" thick):

$V_u \le ØV_n = Ø(V_c + V_s)$, so $V_s \ge \frac{(V_u - ØV_c)}{Ø}$

where V_C = 75 kips

Therefore shear reinforcement is required to resist $V_s \ge \frac{(V_u - ØV_c)}{Ø} = \frac{(102 - 0.75 \times 75)}{0.75} = 61\ kips.$

This shear reinforcement will be provided in the form of 3/8" diameter shear studs (area = 0.11 sqin each) at the maximum spacing of s = d/2 ≅ 4 in. along vertically oriented stud rails. The stud rails will be spaced 16 inches apart horizontally across the width of the wall. In a wall width of 5.67' = 68", there will be 68/16 = 4.25 studs in each horizontal row. Therefore the shear capacity of the shear studs in a 5.67' width of wall is given by ACI-318 as:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.75}{4}$ = 61 Kips

where: $n = \frac{Wall\ width}{Rail\ Spacing} = \frac{68"}{16"}$ = 4.25

A_v = cross-sectional area of 3/8" stud = 0.11 in^2

s_{max} = d/2 = 8.75/2 = 4.375 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_{s\,(needed)}} = \frac{4.25 \times 0.11 \times 60 \times 8.75}{61}$ = 4.1 in

$\therefore s_{used} = 4$ in

Finally, $\text{Ø}V_n = \text{Ø}(V_C + V_S) = 0.75\ (75 + 62) = 103$ Kips > $V_{Tsu} = 102$ Kips, therefore the shear wall is now adequate for transverse shear due to log debris impact.

At mid-height of the wall, $V_{Tsu\ @\ mid\text{-}height} = 63$ Kips > $\text{Ø}V_c = 56$, therefore the stud rails are required up the entire height of shear wall below the inundation depth, h_{max}. It is not possible to reduce the shear capacity by increasing the stud spacing because they are already close to the maximum spacing. Therefore the new shear wall will require 3/8 headed studs at 4 inches on center on stud rails spaced every 16 inches horizontally, up the entire inundated height of the wall.

The original shear wall cross-section at the ground floor of the Seaside example is shown in **Figure 44.** The new wall cross-section designed to resist the tsunami loads is shown in **Figure 45.**

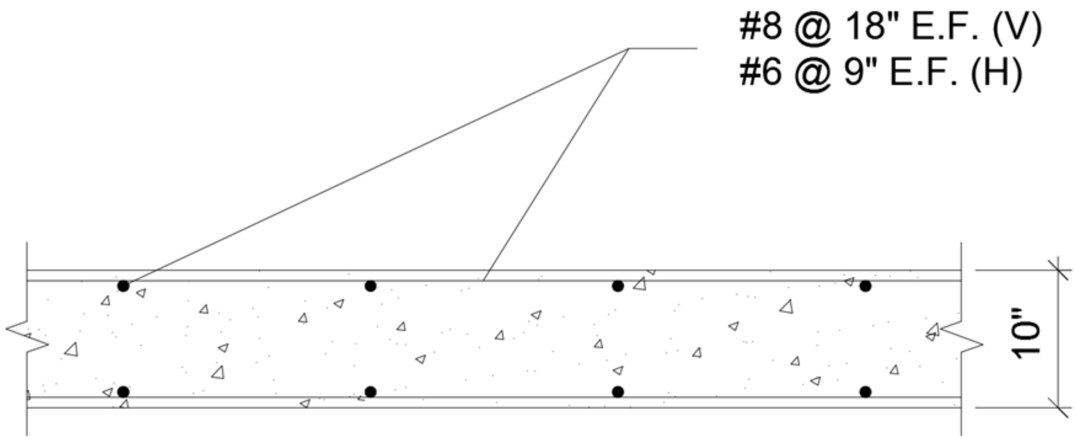

Figure 44: Segment of original exterior wall cross-section at the ground floor level based on SDC D design.

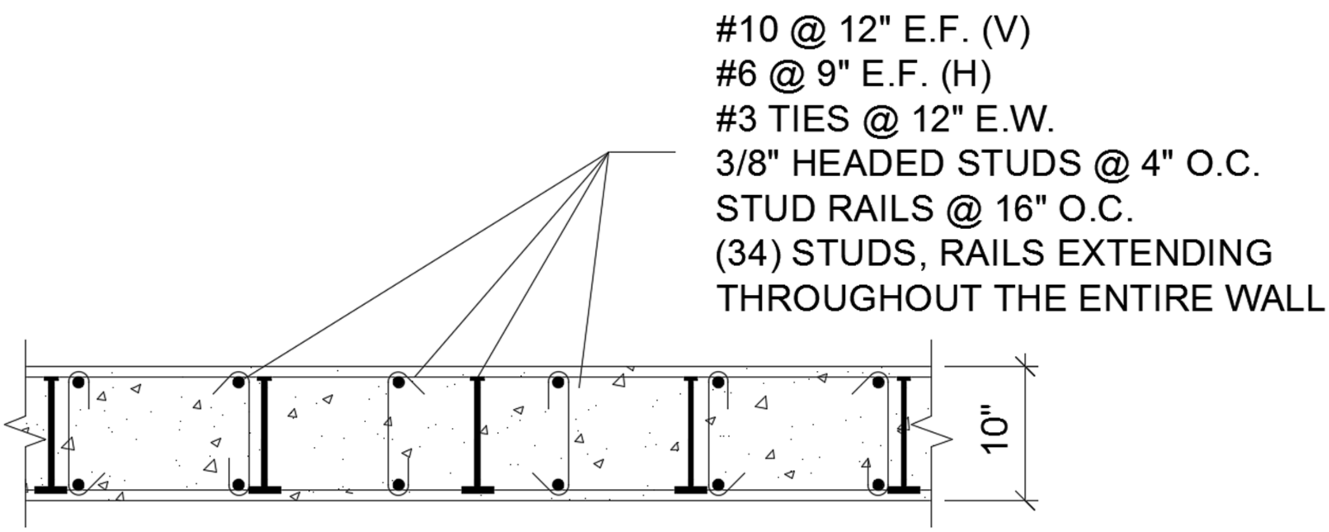

Figure 45: New exterior wall, cross-section at the ground floor level based on tsunami design requirements

Columns Design for Tsunami Component Loads

All of the columns in the residential building are gravity load columns and do not participate in the lateral load resistance. The exterior columns must be designed for hydrodynamic and debris impact component loads, while the interior columns only require design for the hydrodynamic component loads.

Exterior gravity load columns

Exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** requires that C_d = 2.0 and the width dimension, *b*, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Therefore *b* = 0.70x28' = 19.6 ft.

The controlling load case is LC2, when the inundation depth is h_e = 20.93 ft and u_{max} = 37.92 fps.

The hydrodynamic drag is computed using **Eqn 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(20.93 \times 37.92^2)/1000 = 1{,}298\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 1,298/20.93 = 62 kips/ft over the lower 20.93 feet of the column (**Figure 46**). The column must be designed for this load combined with gravity loads per **Section 6.8.3.3**.

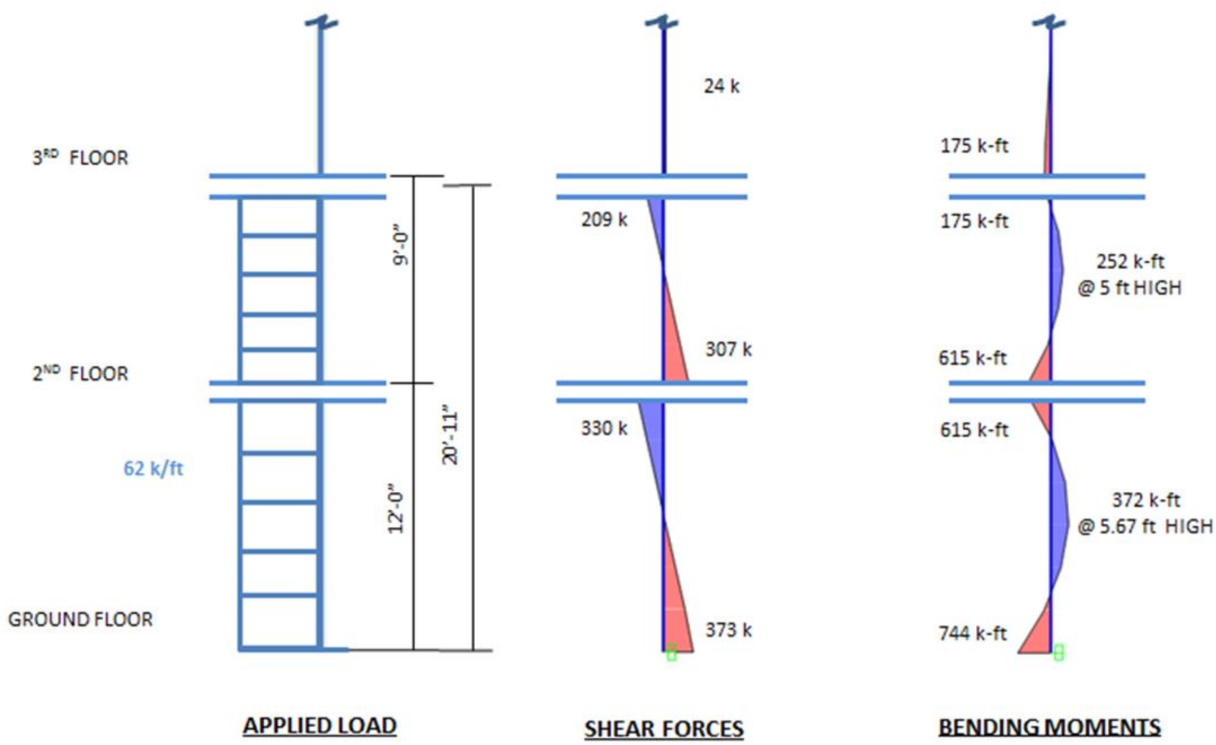

Figure 46: Hydrodynamic loading on exterior column of Seaside residential building due to Load Case 2

The exterior columns must also be designed for log impact load of 107.25 kips at locations that result in maximum bending moment and shear forces. **Figure 47** through **Figure 49** show samples of these loading conditions for a typical exterior column. The impact load is applied at a distance "*d*" (the effective depth of the column longitudinal reinforcement) from the end of the column to evaluate the shear reinforcement in the confined end zone. The impact load is also applied at a distance "$d+h_c$" from the end of the column to evaluate the shear reinforcement immediately outside of the confined section of column with length h_c, equal to the column cross-section largest dimension.

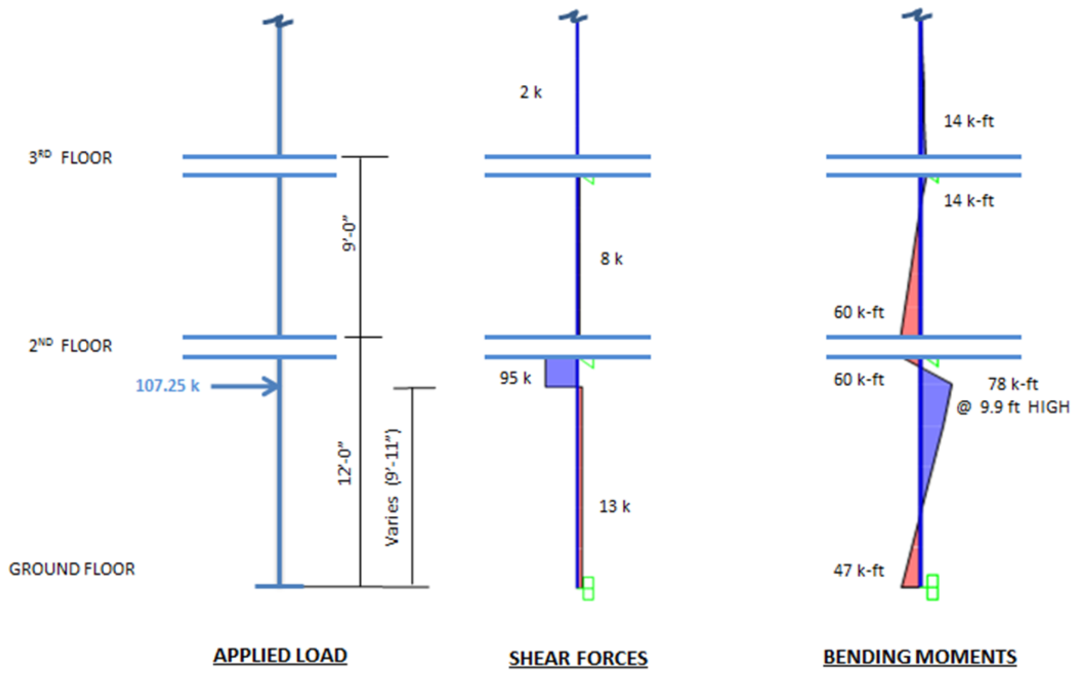

Figure 47: Impact load applied at "*d*" away from the top of column on the ground floor of Seaside residential building

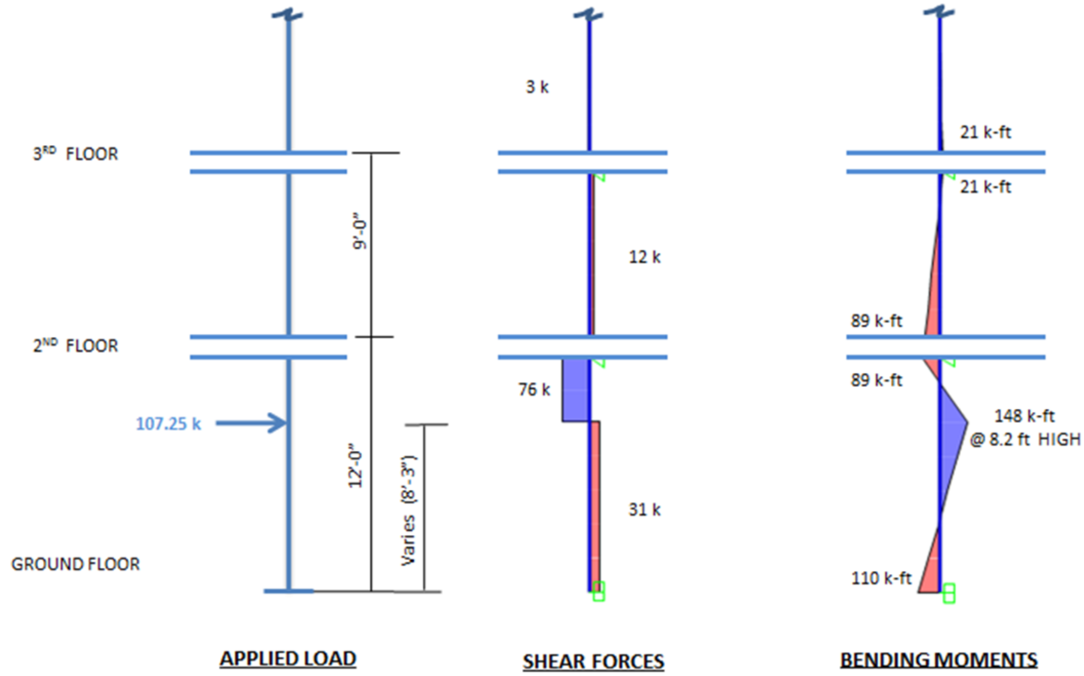

Figure 48: Impact load applied at "$d + h_c$" away from the top of column on the ground floor of the Seaside residential building

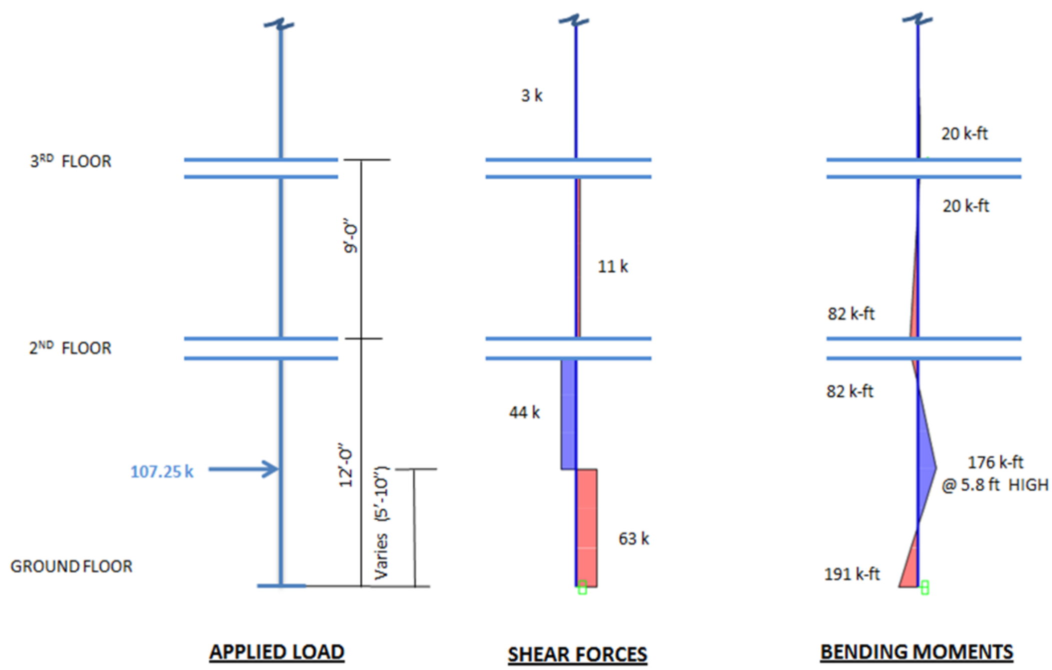

Figure 49: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

The resulting component shear forces and moments for the exterior column are tabulated for every floor in **Table 8**.

Table 8: Results from loading conditions of Seaside residential building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
744	514.3	282	179	1.2D+Ftsu+0.5L (Hydro)
744	365.4	282	179	0.9D+Ftsu (Hydro)
191	514.3	95	76	1.2D+Ftsu+0.5L (Impact)
191	365.4	95	76	0.9D+Ftsu (Impact)
Floor 2				
615	440.8	217	113	1.2D+Ftsu+0.5L (Hydro)
615	313.2	217	113	0.9D+Ftsu (Hydro)
168	440.8	92	70	1.2D+Ftsu+0.5L (Impact)
168	313.2	92	70	0.9D+Ftsu (Impact)
Floor 3				
175	367.4	24	24	1.2D+Ftsu+0.5L (Hydro)
175	261	24	24	0.9D+Ftsu (Hydro)
167	367.4	92	69	1.2D+Ftsu+0.5L (Impact)
167	261	92	69	0.9D+Ftsu (Impact)
Floor 4				
41	293.9	6	6	1.2D+Ftsu+0.5L (Hydro)
41	208.8	6	6	0.9D+Ftsu (Hydro)
94	293.9	92	15	1.2D+Ftsu+0.5L (Impact)
94	208.8	92	15	0.9D+Ftsu (Impact)
Floor 5				
10	220.4	1	1	1.2D+Ftsu+0.5L (Hydro)
10	156.6	1	1	0.9D+Ftsu (Hydro)
31	220.4	3	3	1.2D+Ftsu+0.5L (Impact)
31	156.6	3	3	0.9D+Ftsu (Impact)
Floor 6				
2	146.9	0	0	1.2D+Ftsu+0.5L (Hydro)
2	104.4	0	0	0.9D+Ftsu (Hydro)
7	146.9	1	1	1.2D+Ftsu+0.5L (Impact)
7	104.4	1	1	0.9D+Ftsu (Impact)
Floor 7				
1	73.5	0	0	1.2D+Ftsu+0.5L (Hydro)
1	52.2	0	0	0.9D+Ftsu (Hydro)
2	73.5	0	0	1.2D+Ftsu+0.5L (Impact)
2	52.2	0	0	0.9D+Ftsu (Impact)

The original exterior column design is shown in **Figure 50** and **Figure 51** for the end and center sections, respectively. Column interaction diagrams were created for each exterior column to determine the column design needed to withstand the component moment loads applied to the exterior columns. **Figure 52** shows an integration diagram with each column load condition. The solid blue line represents the original column which can withstand the impact loads, but not the hydrodynamic loads. The green dashed line represents the strengthened column to account for the component loading for both debris impact and hydrodynamic loading.

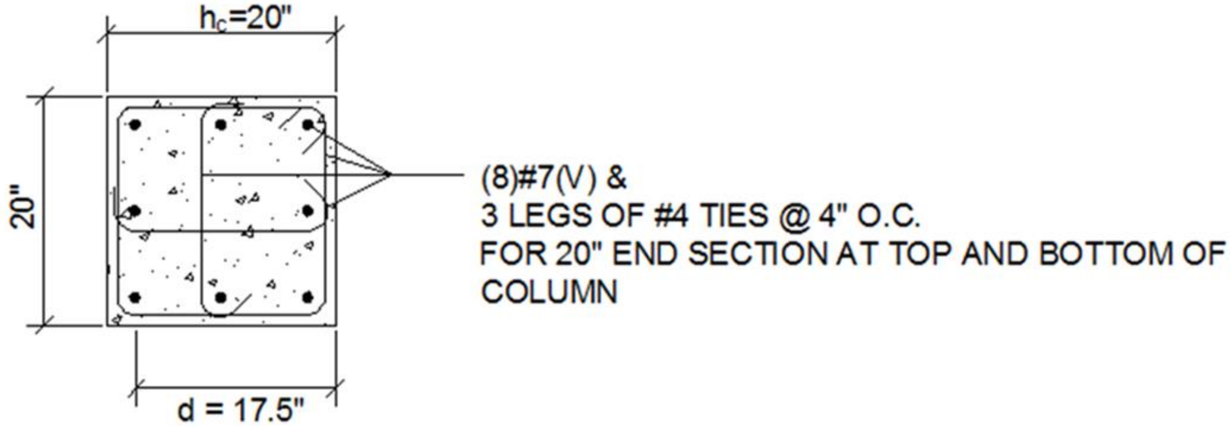

Figure 50: Exterior column cross-section at end of column at all floor levels based on SDC D design

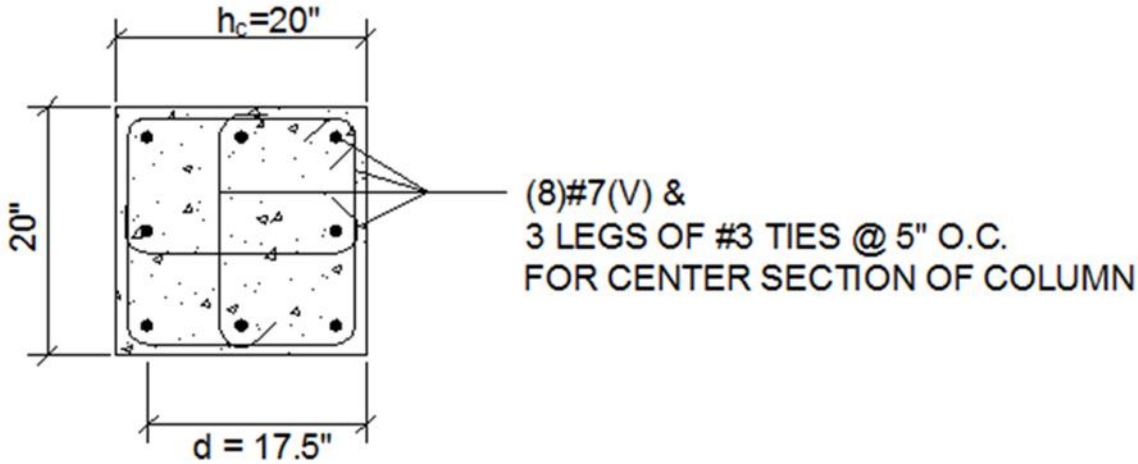

Figure 51: Exterior column cross-section at center of column at all floor levels based on SDC D design

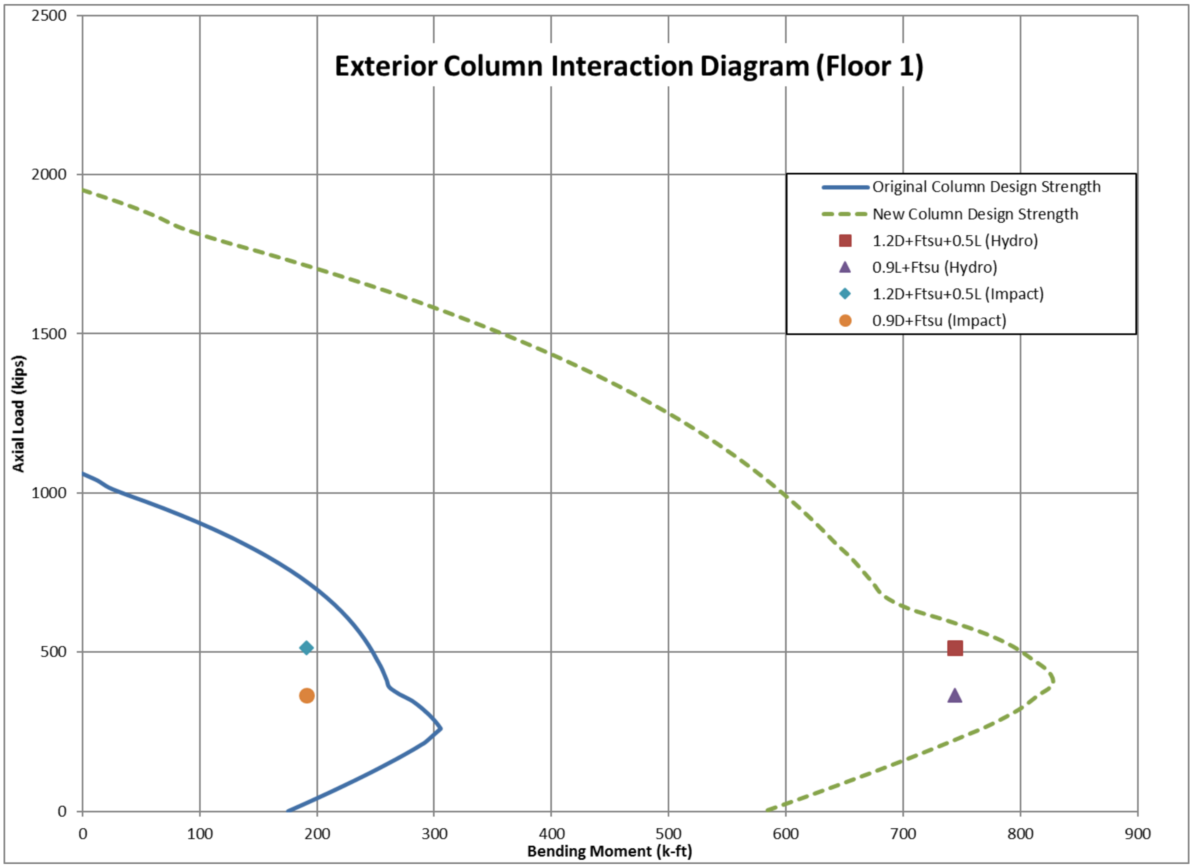

Figure 52: Interaction diagram for typical exterior ground floor column in the Seaside residential building

The new column cross-sections at the end and center of the column are shown in **Figure 53** and **Figure 54**, respectively. It was necessary to increase both the column size and the longitudinal reinforcement to resist the tsunami component loads.

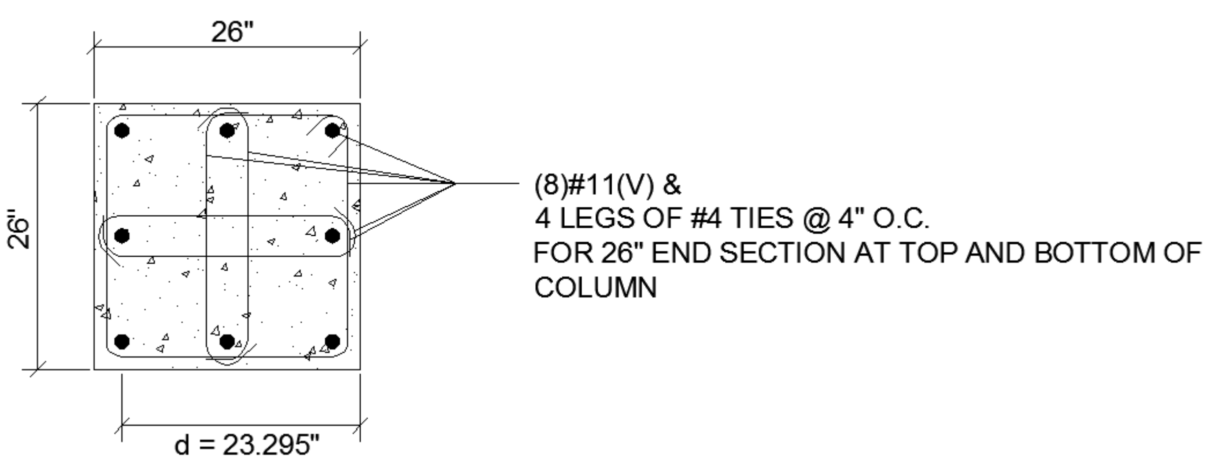

Figure 53: Exterior column cross-section at end of column at ground floor level based on tsunami design requirements

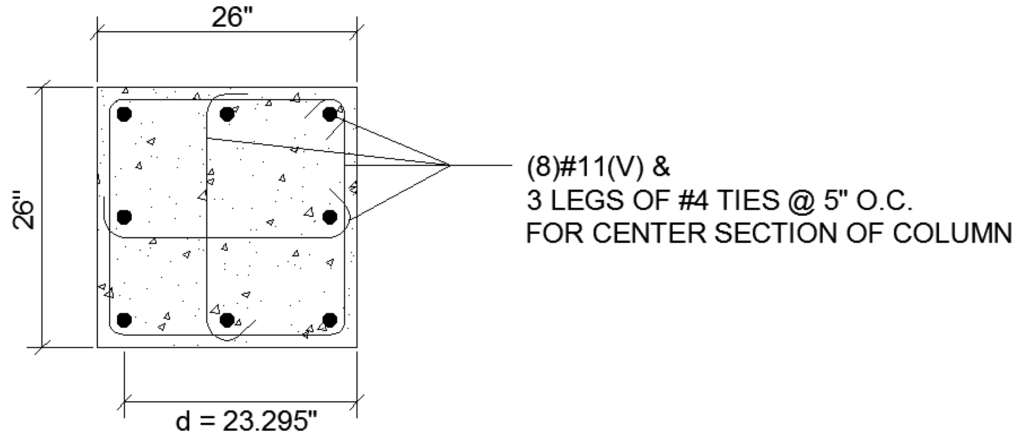

Figure 54: Exterior column cross-section at center of column at ground floor level based on tsunami design requirements

The new exterior columns must also withstand the shear loading due to component hydrodynamic drag and debris impact loading. Based on ACI 318 shear design, the shear capacity is given by:

$$\text{Ø}V_n = \text{Ø}(V_C + V_S)$$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4{,}000}\left(1+\frac{365{,}400}{2{,}000\times26\times26}\right)26\times23.295/1{,}000$ = 97 kips.

In the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times23.295}{4\times1{,}000}$ = 280 kips.

$$V_S = \frac{A_v f_y d}{s} = 280 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4{,}000}\times26\times23.295 = 306 \text{ kips} \therefore \text{use } 280 \text{ kips}$$

In the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times23.295}{5\times1{,}000}$ = 168 kips.

$$V_S = \frac{A_v f_y d}{s} = 168 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4{,}000}\times26\times23.295 = 306 \text{ kips} \therefore \text{use } 168 \text{ kips}$$

Therefore in the end sections, $\text{Ø}V_n$ = 0.75 (97 + 280) = 283 k

and in the center sections, $\text{Ø}V_n$ = 0.75 (97 + 168) = 199 k

At d from the end of the column, V_u = 282 k < ϕV_n = 283 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$ from the end of the column, V_u = 179 k < ϕV_n = 199 k, therefore the column is adequate for shear at the center section.

$$\text{Ø}V_n = \text{Ø}(V_C + V_S)$$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{365{,}400}{2{,}000\times26\times26}\right)26\times23.295/1{,}000$ = 97 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times23.295}{4\times1{,}000}$ = 280 kips

$V_S = \frac{A_v f_y d}{s}$ = 280 kips $\ngtr 8\sqrt{f_C{}'}$ b d $= 8 \times \sqrt{4{,}000} \times 26 \times 23.295 = 306$ kips $\therefore$ use 280 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times23.295}{5\times1{,}000}$ = 168 kips.

$V_S = \frac{A_v f_y d}{s}$ =168 kips $\ngtr 8\sqrt{f_C{}'}$ b d $= 8 \times \sqrt{4{,}000} \times 26 \times 23.295 = 306$ kips $\therefore$ use 168 kips

Therefore in the end sections, $\emptyset V_n$ = 0.75 (97 + 280) = 283 k

and in the center sections, $\emptyset V_n$ = 0.75 (97 + 168) = 199 k

At d: V_u = 282 k < ϕV_n = 283 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: V_u = 179 k < ϕV_n = 199 k, therefore the column is adequate for shear at the center section.

Interior gravity load columns:

Interior columns are 20" (1.67 ft) square R.C. columns. The controlling load case will be LC2, when the inundation depth is h_e = 20.93 ft and u_{max} = 37.92 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**) and b = 1.67 ft since no debris accumulation is considered for interior columns.

Therefore, $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 1.67(20.93 \times 37.92^2)/1000 = 110.4\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 110.4/20.93 = 5.27 kips/ft over the lower 20.93 feet of the column. This load must be combined with gravity loads per **Section 6.8.3.3** and the column capacity verified. The interior columns were adequate as originally designed, so no changes were required to satisfy the tsunami loading conditions.

Cost Implications

Cost Implications for the Office Buildings

As demonstrated in this report for the Seaside office building, the exterior seismically designed moment resisting frame columns provide considerable resistance to the tsunami loads, but still require some increase in reinforcing steel at the first and second floors in order to resist the tsunami loads (see Appendix A). Similar observations were noted at the other three locations. Because of the lower tsunami loading at Monterey CA, there was very little increase in the column reinforcement (see Appendix B). At the Hilo HI, location the tsunami maximum inundation depth is 55.1 feet (16.8 m), resulting in significant increases in both column size and reinforcement requirements for the first five floors of this building (See Appendix C). At the Waikīkī HI location, the flow conditions are only slightly greater than those at the Monterey CA location. However, the lower seismic design requirements (SDC C) result in less robust columns requiring an increase in both column size and reinforcement for the first two floors of the Waikīkī building (see Appendix D).

Based on a prior study by Nitta and Robertson (2012) the overall construction cost of the prototypical 6-story office buildings are estimated as $25,212,500 for the SMRF building at the Seaside, Monterey and Hilo locations and $23,020,400 for the IMRF building at the Waikīkī location. This includes the structural frame and all non-structural components of the building. Note that this study ignores the variation in construction costs between these four locations so as to provide an unbiased comparison of the premium paid for incorporating tsunami design.

The increase in concrete volume and reinforcing steel weight was computed for each building at each of the four sites. Appendix E shows the material quantity calculations for the Seaside location for both Office and Residential buildings.

The resulting increases in concrete materials (including formwork, labor, etc.) and reinforcing steel (including fabrication and installation) for the prototypical Office buildings at each location are shown in **Table 9** through **Table 12**.

Table 9: Material quantity and cost increases for the Office building at the Seaside location.

Seaside Office Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel				
	lb	lb	lb	$
1	15,512	47,636	32,124	48,185
2	10,469	24,498	14,029	21,043
Total	25,982	72,134	46,152	69,229
Shear Steel				
	lb	lb	lb	$
1	7,675	17,756	10,081	15,122
2	6,979	15,996	9,017	13,526
Total	14,653	33,752	19,098	28,648
Concrete				
	yd^3	yd^3	yd^3	$
Total	0	0	0	0
Total				97,876

Table 10: Material quantity and cost increases for the Office building at the Monterey location.

Monterery Office Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel				
	lb	lb	lb	$
1	15,512	28,581	13,069	19,604
2	10,469	19,944	9,475	14,212
Total	25,982	48,526	22,544	33,816
Shear Steel				
	lb	lb	lb	$
1	7,675	22,272	14,597	21,895
2	6,979	6,979	0	0
Total	14,653	29,250	14,597	21,895
Concrete				
	yd^3	yd^3	yd^3	$
Total	0	0	0	0
Total				55,711

Table 11: Material quantity and cost increases for the Office building at the Hilo location.

Hilo Office Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel				
	lb	lb	lb	$
1	15,512	85,882	70,369	105,554
2	10,469	53,184	42,715	64,073
3	10,469	46,536	36,067	54,101
4	10,469	13,296	2,827	4,240
Total	46,920	198,899	151,978	227,967
Shear Steel				
	lb	lb	lb	$
1	7,675	39,191	31,516	47,274
2	6,979	34,386	27,407	41,110
3	6,979	37,151	30,172	45,258
Total	21,632	110,727	89,095	133,642
Concrete				
	yd^3	yd^3	yd^3	$
1	90	184	94	141
2	77	158	81	121
3	77	158	81	121
Total	245	500	255	383
Total				361,993

Table 12: Material quantity and cost increases for the Office building at the Waikīkī location.

Waikiki Office Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel				
	lb	lb	lb	$
1	9,649	38,109	28,459	42,689
2	8,271	12,406	4,135	6,203
Total	17,920	50,515	32,595	48,892
Shear Steel				
	lb	lb	lb	$
1	2,085	23,564	21,479	32,219
2	1,828	2,148	320	480
Total	3,913	25,712	21,799	32,699
Concrete				
	yd^3	yd^3	yd^3	$
1	66	104	37	56
Total	66	104	37	56
Total				81,647

Cost Implications for the Residential Buildings

As demonstrated in this report for the Seaside residential building, the exterior gravity load columns require increases in cross-section size and in reinforcing steel at the first and second floors in order to resist the tsunami loads. In addition, the shear walls located on the exterior of the building are exposed to debris impact which requires the addition of headed studs to improve the out of plane shear capacity (See Appendix A). Similar observations were noted at the other three locations. Because of the lower tsunami loading at Monterey CA, there was very little increase in the column reinforcement (See Appendix B). At the Hilo HI, location the tsunami maximum inundation depth is 55.1 feet (16.8 m), resulting in significant increases in both column and shear wall size and reinforcement requirements for the first five floors of this building (See Appendix C). At the Waikīkī HI, location, the flow conditions are only slightly greater than those at the Monterey CA, location. The lower seismic design requirements (SDC C) result in less robust shear walls requiring an increase in both wall thickness and reinforcement, while the gravity load exterior columns require strengthening of the first two floors similar to the Seaside building (See Appendix D).

Based on a prior study by Nitta and Robertson (2012) the overall construction cost of the prototypical buildings are estimated as $23,305,700 for the special shear wall building at the Seaside, Monterey and Hilo locations and $22,021,300 for the ordinary shear wall building at the Waikīkī location. This includes the structural frame and all non-structural components of the building. Note that this study ignores the variation in construction costs between these four locations so as to provide an unbiased comparison of the premium paid for incorporating tsunami design.

The resulting increases in concrete materials (incl. formwork, labor, etc.) and reinforcing steel (incl. fabrication and installation) for the prototypical Residential buildings at each location are shown in Table 14 through 16.

Table 13: Material quantity and cost increases for the Residential building at the Seaside location.

Seaside Residential Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel (Exterior Column)				
	lb	lb	lb	$
1	3,141	8,166	5,025	7,538
2	2,356	3,926	1,570	2,356
Total	5,496	12,092	6,596	9,894
Flexural Steel (Wall)				
	lb	lb	lb	$
1	2,412	5,817	3,405	5,107
2	1,374	3,435	2,061	3,092
3	1,008	3,435	2,428	3,641
Total	4,794	12,688	7,893	11,840
Shear Steel (Exterior Column)				
	lb	lb	lb	$
1	2,455	5,379	2,924	4,386
2	2,002	2,821	819	1,229
Total	4,457	8,200	3,743	5,615
Shear Steel (Wall)				
	lb	lb	lb	$
1	0	5,544	5,544	8,316
2	0	4,032	4,032	6,048
3	0	4,032	4,032	6,048
4	0	1,344	1,344	2,016
Total	0	14,952	14,952	22,428
Concrete				
	yd^3	yd^3	yd^3	$
1	20	33	14	14,925
2	15	25	10	11,194
Total	35	58	24	26,119
Total				75,895

Table 14: Material quantity and cost increases for the Residential building at the Hilo location.

Hilo Residential Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel (Exterior Column)				
	lb	lb	lb	$
1	3,141	13,296	10,155	15,233
2	2,356	6,203	3,847	5,771
3	2,356	5,889	3,533	5,300
4	2,356	5,889	3,533	5,300
Total	10,208	31,277	21,070	31,604
Flexural Steel (Wall)				
	lb	lb	lb	$
1	9,822	21,724	11,902	17,854
2	6,527	11,533	5,006	7,509
3	4,786	9,593	4,806	7,209
4	4,786	7,266	2,480	3,720
5	1,008	2,061	1,053	1,580
6	1,008	2,061	1,053	1,580
Total	27,937	54,238	26,301	39,452
Confining Steel (Wall)				
	lb	lb	lb	$
1	0	4,773	4,773	7,160
2	0	2,240	2,240	3,360
3	0	1,726	1,726	2,590
4	0	636	636	954
Total	0	9,376	9,376	14,064
Shear Steel (Exterior Column)				
	lb	lb	lb	$
1	2,908	10,184	7,276	10,913
2	2,219	3,896	1,678	2,516
3	2,219	4,705	2,486	3,729
4	2,219	4,234	2,015	3,023
Total	9,564	23,019	13,455	20,182
Shear Steel (Wall)				
	lb	lb	lb	$
1	0	2,206	2,206	3,310
2	0	760	760	1,140
3	0	760	760	1,140
4	0	760	760	1,140
5	0	760	760	1,140
6	0	760	760	1,140
Total	0	6,006	6,006	9,010
Concrete (Exterior Column)				
	yd^3	yd^3	yd^3	$
1	20	39	19	20,765
2	15	29	14	15,574
3	15	29	14	15,574
4	15	29	14	15,574
Total	64	126	62	67,487
Concrete (Wall)				
	yd^3	yd^3	yd^3	$
1	36	44	8	8,544
2	27	33	6	6,408
3	27	33	6	6,408
4	27	33	6	6,408
5	16	22	6	6,814
6	16	22	6	6,814
Total	147	185	38	41,395
Total				223,194

Table 15: Material quantity and cost increases for the Residential building at the Monterey location.

Monterey Residential Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel (Exterior Column)				
	lb	lb	lb	$
1	3,141	8,166	5,025	7,538
2	2,356	4,986	2,630	3,946
Total	5,496	13,152	7,656	11,484
Shear Steel (Exterior Column)				
	lb	lb	lb	$
1	2,908	4,659	1,751	2,626
Total	2,908	4,659	1,751	2,626
Shear Steel (Wall)				
	lb	lb	lb	$
1	0	2,940	2,940	4,410
2	0	2,184	2,184	3,276
3	0	252	252	378
Total	0	5,376	5,376	8,064
Concrete (Exterior Column)				
	yd^3	yd^3	yd^3	$
1	20	28	9	9,517
Total	20	28	9	9,517
Total				31,691

Table 16: Material quantity and cost increases for the Residential building at the Waikīkī location.

Waikiki Residential Builidng				
Floor	Original Building	New Building	Δ	Cost
Flexural Steel (Exterior Column)				
	lb	lb	lb	$
1	3,141	6,648	3,507	5,261
2	2,356	3,926	1,570	2,356
Total	5,496	10,574	5,078	7,616
Shear Steel (Exterior Column)				
	lb	lb	lb	$
1	2,908	4,741	1,833	2,750
2	2,219	2,898	679	1,019
Total	5,127	7,639	2,513	3,769
Shear Steel (Wall)				
	lb	lb	lb	$
1	0	2,772	2,772	4,158
2	0	2,016	2,016	3,024
3	0	1,176	1,176	1,764
Total	0	5,964	5,964	8,946
Concrete (Exterior Column)				
	yd^3	yd^3	yd^3	$
1	20	33	14	14,989
2	15	25	10	11,241
Total	35	58	24	26,230
Total				46,562

Table 17 shows a comparison between the original cost of each of the office and residential buildings at the four selected locations with the cost of the buildings if the tsunami design requirements of ASCE/SEI 7-16 are incorporated in the building design. The percentage increase in the overall cost of the buildings is lowest for Monterey, California (0.221% and 0.158% for the office and residential buildings, respectively) and highest for Hilo, Hawaii (2.482% and 1.197% for the office and residential buildings, respectively). This is primarily attributed to the far larger flow depth and velocity required for the design in Hilo. The cost implications of including tsunami design are greater in Waikīkī, Hawaii (0.530% and 0.211% for the office and residential buildings, respectively) than Seaside, Oregon (0.388% and 0.326% for the office and residential buildings, respectively) even though design tsunami flow conditions are more severe in Seaside. This is attributed to the advantages of the high seismic design and detailing required in the Seaside buildings compared with the lower seismic requirements in Waikīkī due to the lower seismic demand. It is also noted that the cost increment for including tsunami design in the residential buildings with shear walls is lower than for the office buildings with moment resisting frames at the same locations. This is attributed to the relative ease of strengthening a limited number of shear walls as opposed to strengthening all of the columns making up the moment resisting frames. The increase in cost of the shear wall buildings would have been even lower if the shear walls had not been located on the exterior of the building, exposing them to debris impact from floating logs.

Table 17: Cost increases due to incorporation of tsunami design for both Office and Residential buildings at all four locations.

	Building Type	New Cost of Building	Original Cost of Building	**% Overall Cost Increase**
Seaside	Office	25,310,334.81	25,212,458.50	0.388
	Residential	23,381,613.64	23,305,718.33	0.326
Monterey	Office	25,268,169.94	25,212,458.50	0.221
	Residential	23,342,464.89	23,305,718.33	0.158
Hilo	Office	25,994,337.23	25,212,458.50	3.101
	Residential	23,584,741.54	23,305,718.33	1.197
Waikiki	Office	23,142,462.37	23,020,401.50	0.530
	Residential	22,067,849.95	22,021,288.33	0.211

Summary and Conclusions

This study applied the new tsunami design provisions of ASCE/SEI 7-16 to two prototypical mid-rise reinforced concrete buildings located near the shoreline in four coastal communities around the US Pacific Coast. The buildings were assumed to be located in Seaside, Oregon, Monterey, California, Hilo, Hawaii and Waikīkī, Hawaii, and were initially designed for the appropriate wind and seismic loading at each of those locations. One building represented a typical 6-story office building with reinforced concrete special moment frames providing lateral load resistance. The other building represented a typical 7-story residential building with reinforced concrete shear walls providing lateral load resistance. It was assumed that the buildings are supported by deep foundations that are able to resist the effects of scour. It was also assumed that the buildings were not within the large debris influence area of ports or shipping container storage yards, so that the design debris impact was due to floating logs.

The following conclusions were drawn from this study:

1) High seismic design provides considerable strength, ductility and toughness for structural elements of reinforced concrete buildings, which in turn provides increased resistance to tsunami loads.
2) For locations with high seismic demand but relatively low tsunami flow depths, such as Monterey, California, there is very little additional strengthening required when incorporating tsunami design (Less than 0.3%).
3) For locations with high seismic demand and also high tsunami flow depths, such as Hilo, Hawaii, there is a more substantial additional cost required to meet the tsunami design requirements (up to 2.5% of the overall building cost).
4) Buildings in areas of low to moderate seismic demand will require greater strengthening to meet the tsunami design requirements, such as Waikīkī, Hawaii where there cost increases were on the order of 0.5%.
5) Exposure to shipping container or large vessel impact will increase these cost increments due to the need for additional shear and flexural strength in the exterior structural elements. However,

much of this cost could be mitigated by considering the impact load as a dynamic impulse, as opposed to the more conservative static load considered in this study.

6) The residential buildings with shear walls provided a more economical means of resisting the tsunami loads than the exterior moment resisting frames in the office buildings. This effect would have been even more pronounced if the shear walls had been located in the interior of the building so that they are not subjected to debris impact strikes.
7) Buildings taller than the 6- and 7-story heights considered in this study will require even less strengthening because of their greater seismic resistance, particularly at the lower levels exposed to tsunami loads.
8) Buildings that are shorter than the 6- and 7-story heights considered in this study would be expected to require greater strengthening to resist the tsunami loads because of the reduced seismic strength in the lower floor structural elements.

This study demonstrates that tsunami design can be incorporated into the design of multistory reinforced buildings without a significant increase in overall building cost. Coastal communities exposed to tsunami hazard are encouraged to require tsunami design of taller buildings so as to provide refuges-of-last-resort for individuals caught in the tsunami inundation zone without time to evacuate horizontally to high ground.

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A. Seaside Design Example – Appendix A

A.1 Project Site

The Seaside design example considers a multi-story reinforced concrete building in Seaside, Oregon, at the location shown in **Figure A-1**. The center of the building footprint is located at 45.99483 N; 123.9295 W, which is 1025 feet from the shoreline. **Figure A-1** also shows the three topographic transects along which the Energy Grade Line Analysis needs to be applied. The center transect, C, is drawn perpendicular to the shoreline, represented by the average coastline for 500 feet either side of the center transect. The clockwise, CW, and counterclockwise, CCW, transects are generated by rotating the center transect through 22.5 degrees in each direction, about the geometric center of the building plan at the grade plane (ASCE 7 Section 6.8.6.1). Each transect is then extended till it reaches the runup points on the ASCE 7 Tsunami Design Zone map. If the end of a transect falls between two of the runup points, then the runup elevations can be interpolated. The resulting runup elevations for each transect are given in **Table A-1** along with the approximate inundation limit distances obtained using Google Earth. These inundation limit distances will be revised once the runup elevations are plotted on the respective topographic profiles.

Note that for the States of Washington, Oregon and California, the ASCE 7 TDZ maps provide the runup elevations in relation to Mean High Water, MHW, and NAVD88. At the Seaside location the difference between these two elevation models is 6.81 feet. This difference varies along the Pacific Coast.

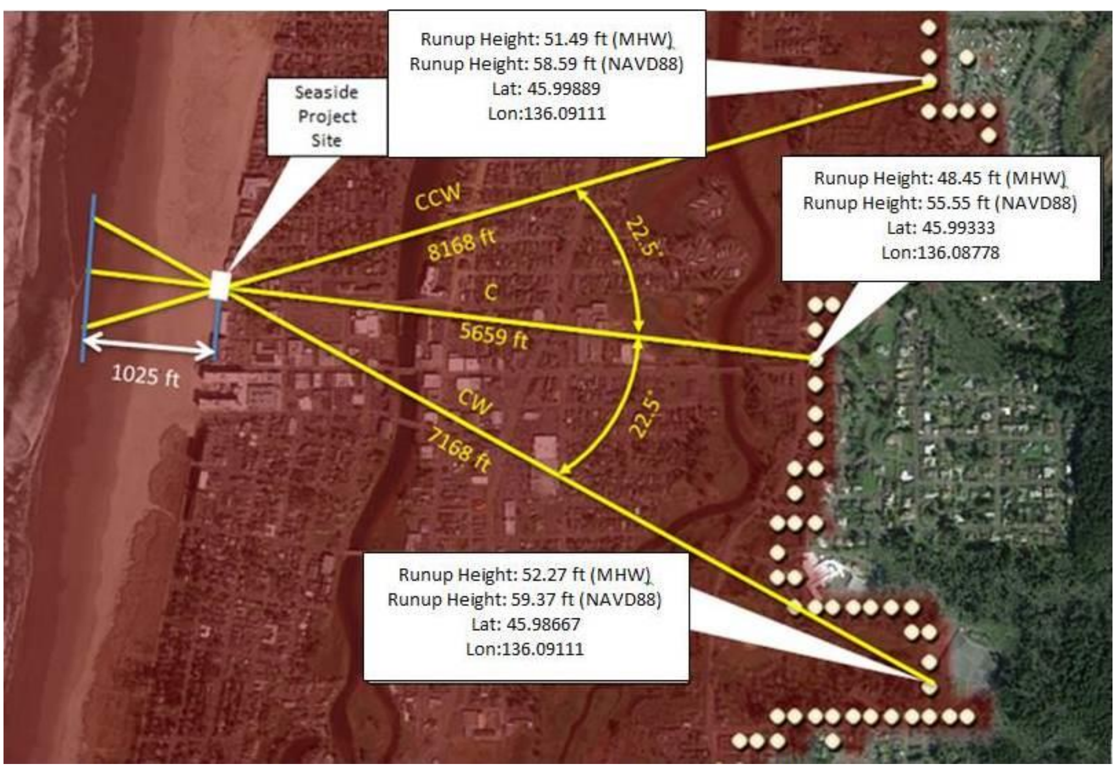

Figure A-1: Location of project site in Seaside, Oregon, relative to inundation line defined by ASCE7-16 Tsunami Design Zone Map. The 22.5° variation in principal flow direction required by Section 6.8.6.1 results in Clockwise (CW) and Counterclockwise (CCW) transects on either side of the Center (C) transect.

Table A-1: Runup elevation and inundation limits for three transects through the Seaside Project site.

Transect	Runup Elevation (ft)				Inundation Limit (ft)	
	MHW Reference		NAVD88 Reference		From TDZ in Google Earth	WGS 84 Reference
	From TDZ	Incl. Sea Level Rise	From TDZ	Incl. Sea Level Rise		
Center	**48.45**	**48.55**	**55.55**	**55.65**	**5640**	**5659**
Counterclockwise	**51.49**	**51.59**	**58.59**	**58.69**	**6707**	**8168**
Clockwise	**52.27**	**52.37**	**59.37**	**59.47**	**7245**	**7168**

A.2 Sea Level Change – Section 6.5.3

ASCE 7 Section 6.5.3 requires that any anticipated sea level rise be included in the runup elevation used in the tsunami design. For this example, we will assume sea level change based on a 50 year project life cycle. ASCE 7 Commentary Section C6.5.3 provides a link to http://tidesandcurrents.noaa.gov/sltrends for historical sea level trends relative to mean sea level (MSL).

From the referenced website the following information is obtained:

"Hammond, OR 9439011

The mean sea level trend is -1.22 mm/year with a 95% confidence interval of +/- 1.81 mm/year based on monthly mean sea level data from 1983 to 2014 which is equivalent to a change of -0.40 feet in 100 years."

The tsunami design should therefore consider the extrapolated prediction of -1.22 + 1.81 = 0.59 mm/year over the 50 year project life cycle. This results in a sea level rise of 29.5 mm or 1.2" (0.10 ft). This must be added to the runup elevation for use in the Energy Grade Line Analysis, as shown in **Table A-1**.

A.3 Topographic Profiles

The topographic profiles along each of these transects was obtained from a Digital Elevation Model, DEM, with the following datums and resolution:

Horizontal Datum: WGS 84

Vertical Datum: MHW

Resolution: 1/3 sec (approximately 10)

The topographic profiles are shown for the Center, Counterclockwise and Clockwise transects in **Figure A-2**, **Figure A-3**, and **Figure A-4** respectively. A horizontal line is plotted on each profile representing the runup elevation (including sea level rise) for each of these transects relative to the MHW datum from

Table A-1. The point where this line intersects the profile represents the inundation limit and the starting point for the Energy Grade Line Analysis. The resulting inundation limit should be cross-checked with the Tsunami Design Zone map inundation line to ensure that they are similar distances from the shoreline (See **Table A-1**). If the TDZ inundation is significantly greater than the first intersection of the runup elevation line with the topographic profile, it may indicate that a region of high ground is present in the inundation zone. The runup elevation must then be modified to match this high ground elevation and the corresponding inundation limit determined where the modified runup elevation next intersects the topographic profile. The resulting values for inundation limit are shown in **Table A-1** and are used in the EGLA along each transect.

The project site location is also indicated on each plot. For the center transect, the site is located 1,025 feet from the shoreline (**Figure A-2**). The elevations at the project site vary slightly for the three transects, which can be attributed to slight differences in the elevation data points used to generate each transect profile.

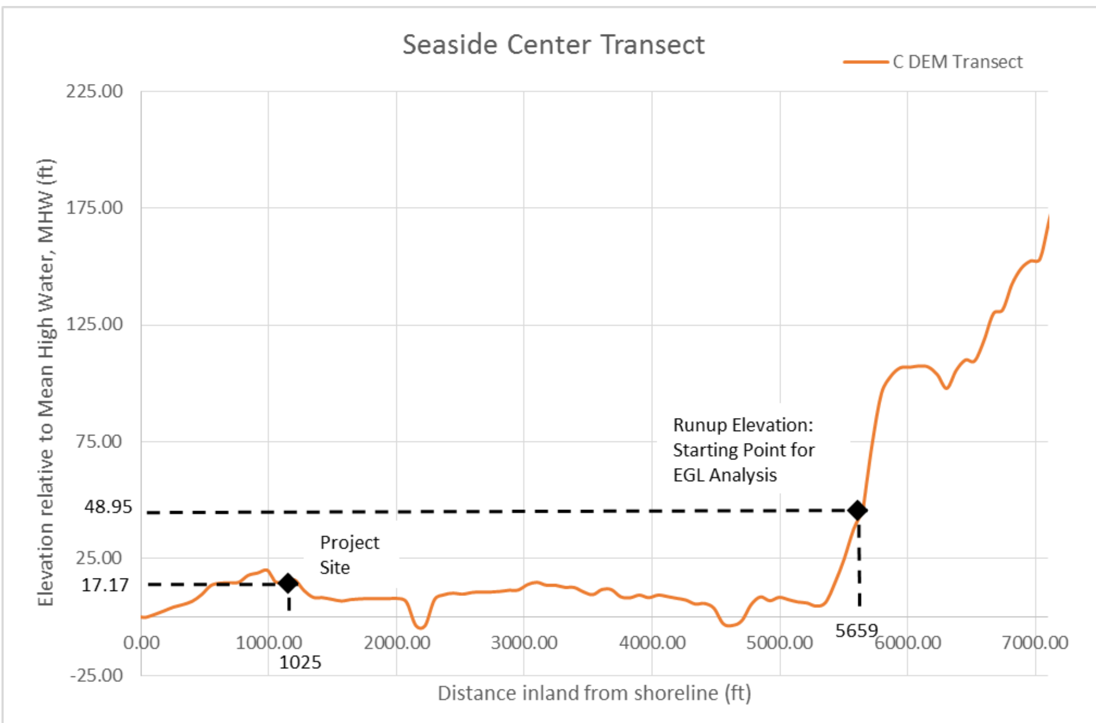

Figure A-2: Topographic profile for Center transect

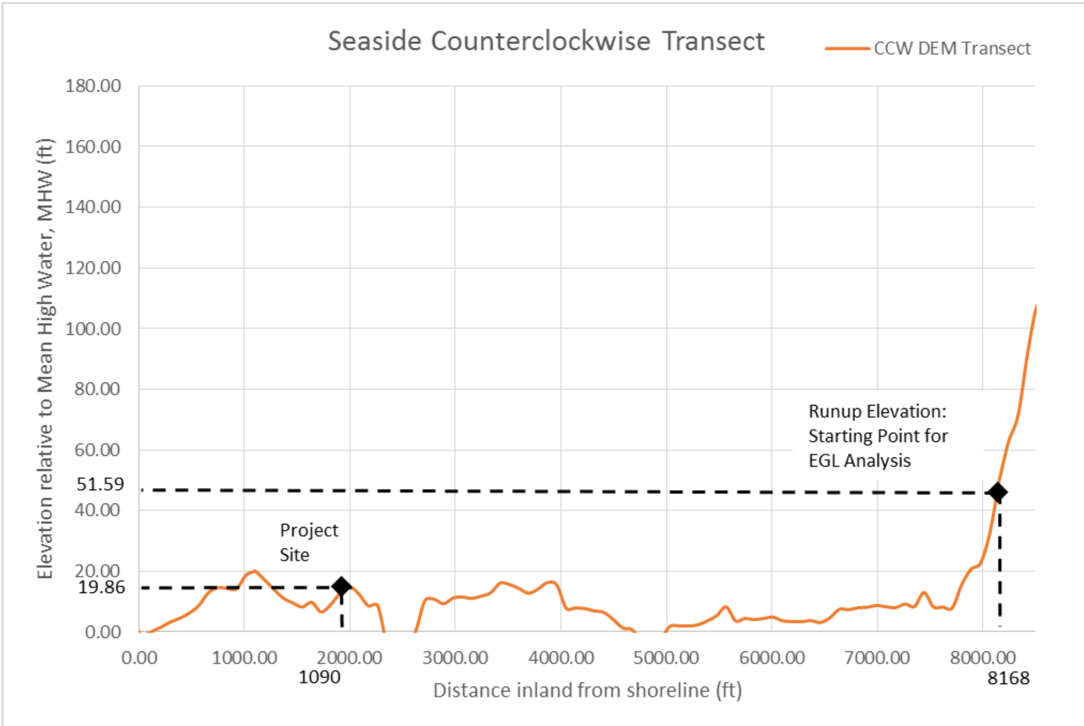

Figure A-3: Topographic profile for Counterclockwise transect

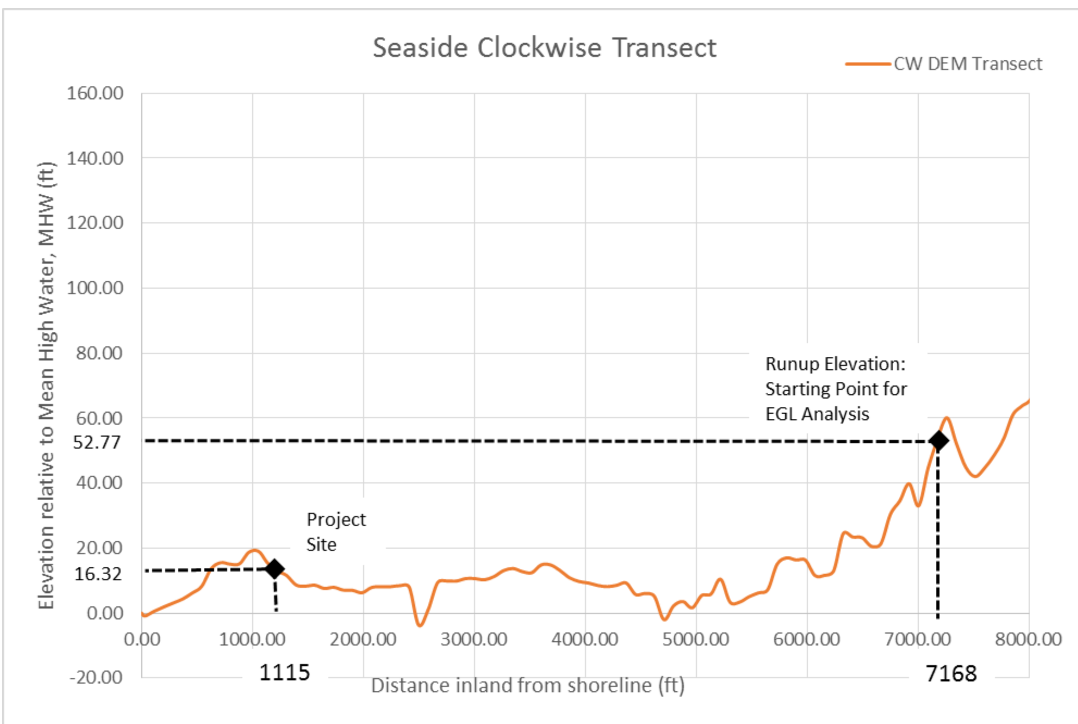

Figure A-4: Topographic profile for Clockwise transect

A.4 Tsunami Bore Determination

In order to determine whether or not a tsunami bore must be considered at the project site, the conditions in ASCE 7 Section 6.6.4 are evaluated for each transect. Tsunami bores shall be considered where any of the following conditions exist:

1. Prevailing nearshore bathymetric slope is 1/100 or milder – YES (See **Figure A-5** and associated discussion).
2. Shallow fringing reefs or other similar step discontinuities – Does not apply.
3. Where historically documented – Does not apply.
4. As described in the Recognized Literature – Does not apply.
5. As determined by a site-specific inundation analysis – not required for TRC II buildings.

Therefore bore loading must be considered in this design.

Figure A-5 shows the approach to determining the average nearshore bathymetric slope so as to determine whether or not tsunami bores need to be considered per ASCE 7 Section 6.4.4. A central line is drawn perpendicular to the shoreline. This line is an extension of the center transect running through the project site. The distance from the shoreline to the 100 meter bathymetric line, indicated by the offshore data points in the ASCE offshore wave maps, is then used to determine the average nearshore bathymetric slope. If any of the transect lines does not intersect the 100 meter bathymetric line, this transect can be ignored for the purpose of determining whether or not there is a bore.

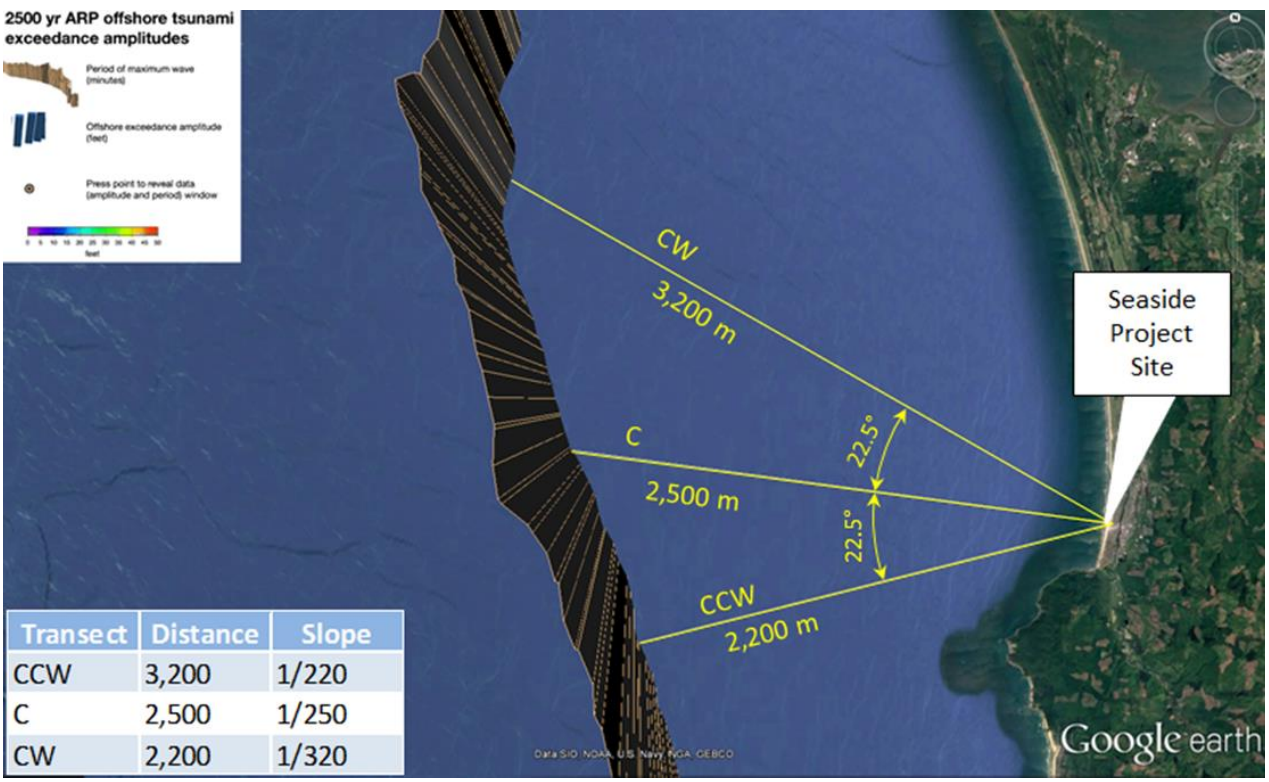

Figure A-5: Determination of average nearshore slope from 100 meter bathymetric line to shoreline along a line perpendicular to the shoreline and lines rotated 22.5 degrees to either side of the center line.

The average nearshore bathymetric slope is then computed using:

$$\emptyset = \frac{100}{distance} \quad \text{in meters}$$

or

$$\emptyset = \frac{328}{distance} \quad \text{in feet .}$$

The table in **Figure A-5** shows that the near shore slope is milder than 1/100, therefore this project site would create bores through prevailing nearshore bathymetric slope.

A.5 Determination of Inundation Depth and Flow Velocity using EGLA

The Energy Grade Line Analysis (EGLA) is a stepwise procedure starting from the run up elevation at the mapped inundation limit, and working shoreward to get the flow parameters at the site of interest.

A spreadsheet was used to perform this operation along all three transects. The input values were the runup, including sea level rise, referenced to MHW datum (**Table A-1** column 3), the inundation limit distance determined from the topographic profile (**Table A-1** column 5), a Manning's coefficient of 0.030 representing "all other cases" from ASCE 7 Table 6.6-1, and α = 1.3 representing bore conditions at the shoreline as specified in ASCE 7 Section 6.6.4. The resulting inundation depth profiles, both with and without the topographical elevation, are shown in **Figure A-6** and **Figure A-7** for the Center transect, **Figure A-8** and **Figure A-9** for the Counterclockwise transect, and **Figure A-10** and **Figure A-11** for the Clockwise transect.

The Clockwise transect results in the largest flow depth of 31.4 feet at the project site, which is the value of h_{max} that will be used in the subsequent design calculations.

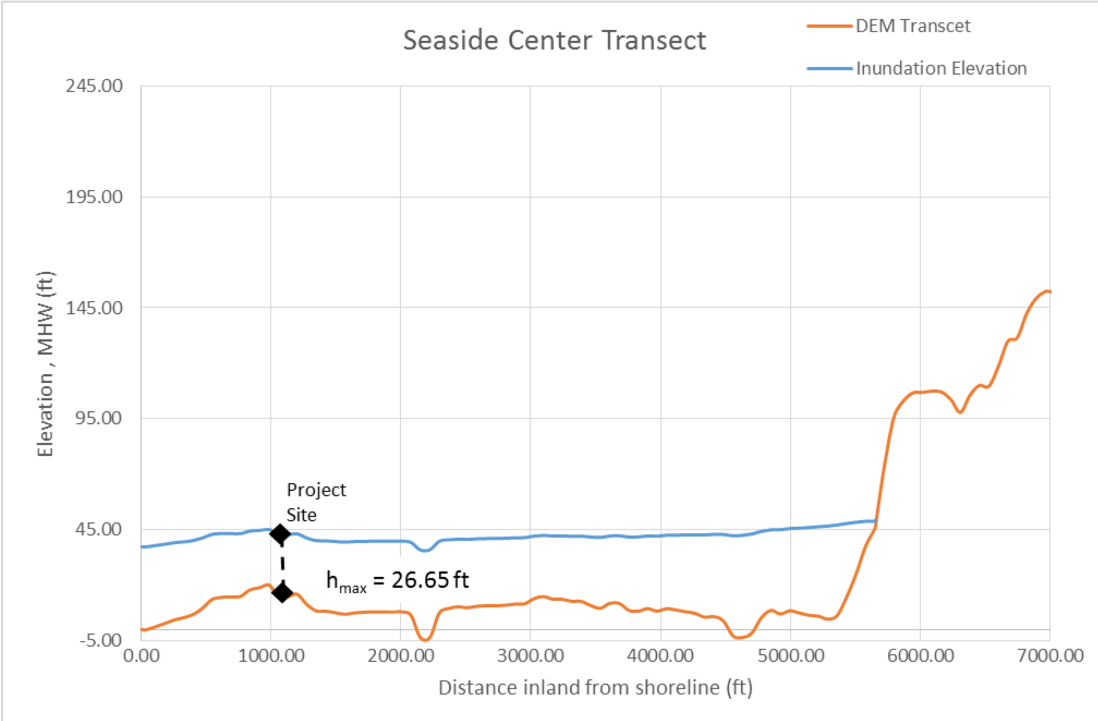

Figure A-6: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for center transect

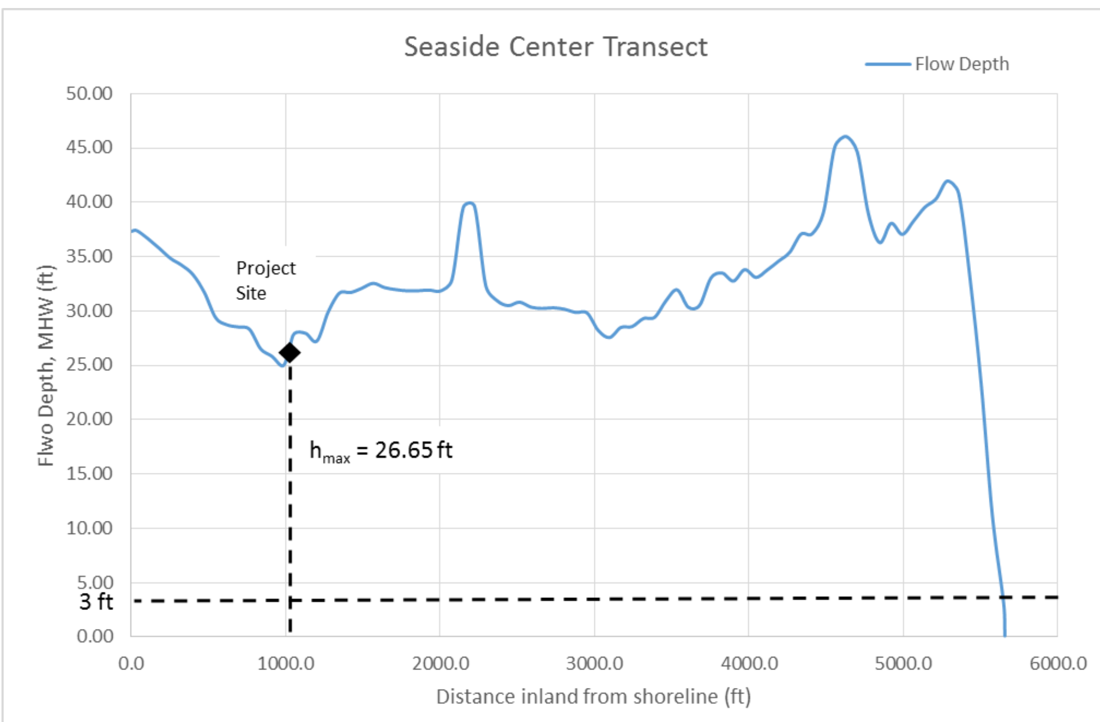

Figure A-7: Inundation depth (h_i) profile from Energy Grade Line analysis for center transect

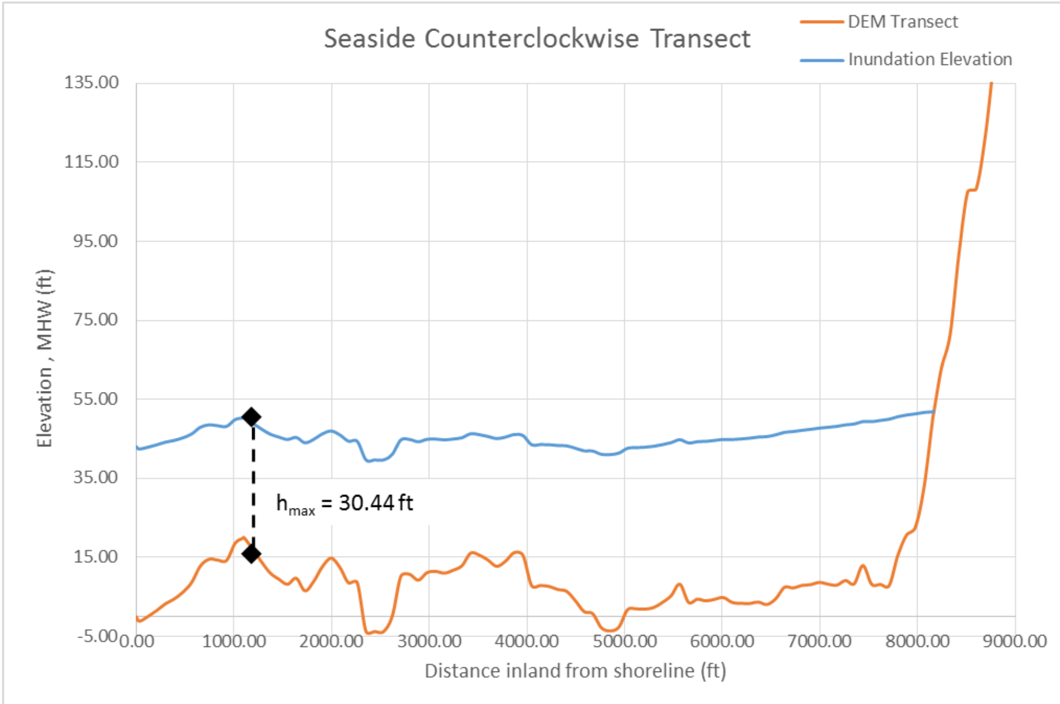

Figure A-8: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for counterclockwise transect

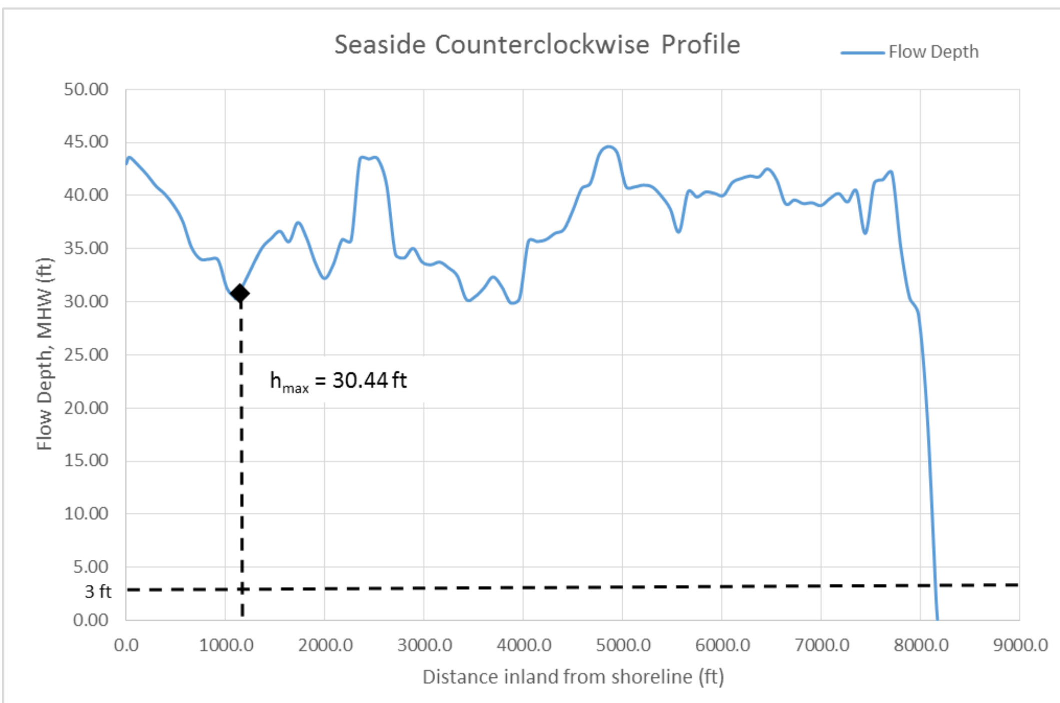

Figure A-9: Inundation depth (h_i) profile from Energy Grade Line analysis for counterclockwise transect

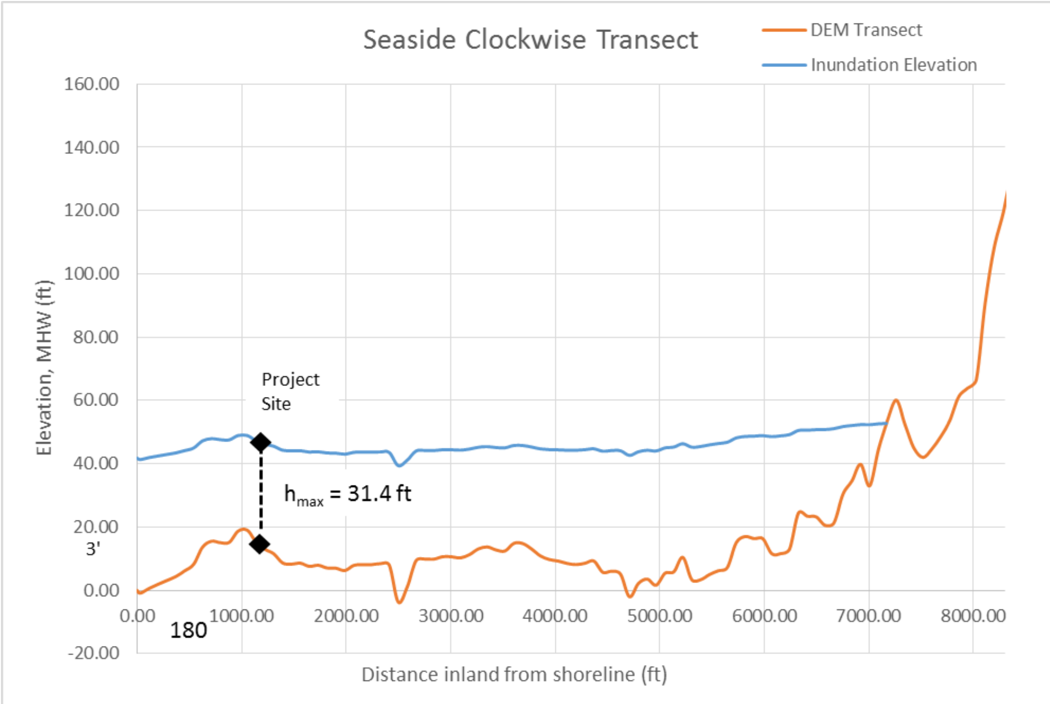

Figure A-10: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for clockwise transect

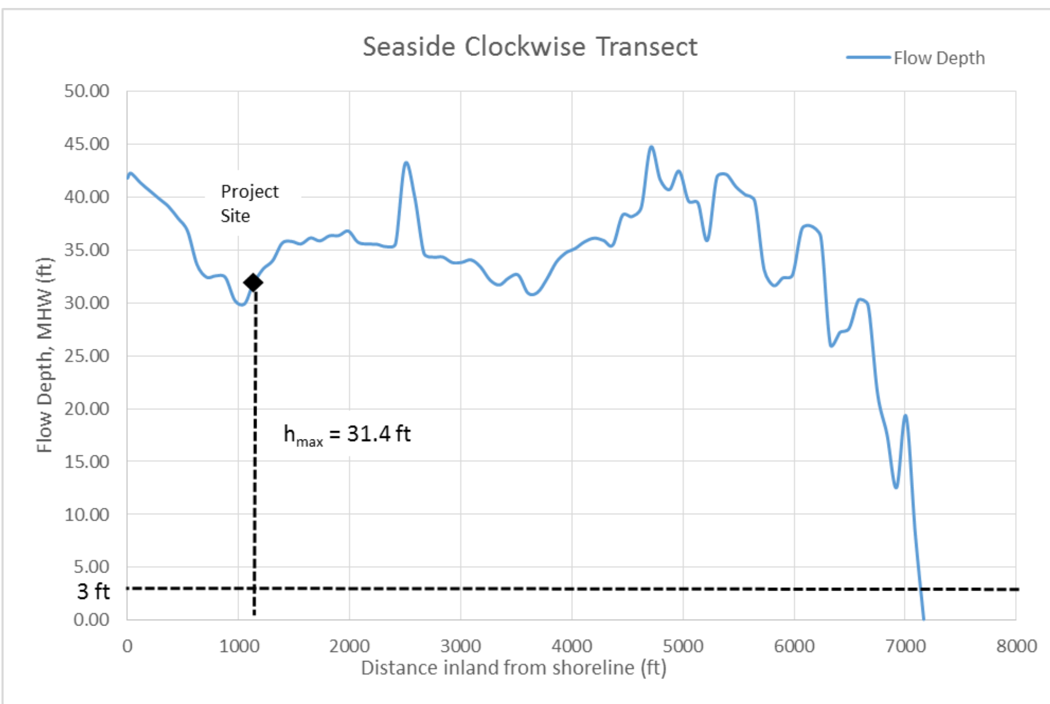

Figure A-11: Inundation depth (h_i) profile from Energy Grade Line analysis for clockwise transect

The flow velocity profiles across each transect as determined from the EGLA are shown in **Figure A-12**, **Figure A-13** and **Figure A-14** for the Center, Counterclockwise and Clockwise transects, respectively. The minimum flow velocity that may be considered is 10 ft/sec, which is indicated on each of the plots. As with the flow depth, the Clockwise transect produces the largest estimate of flow velocity at 37.92 ft/sec, which is the value of u_{max} that will be used in the design calculations.

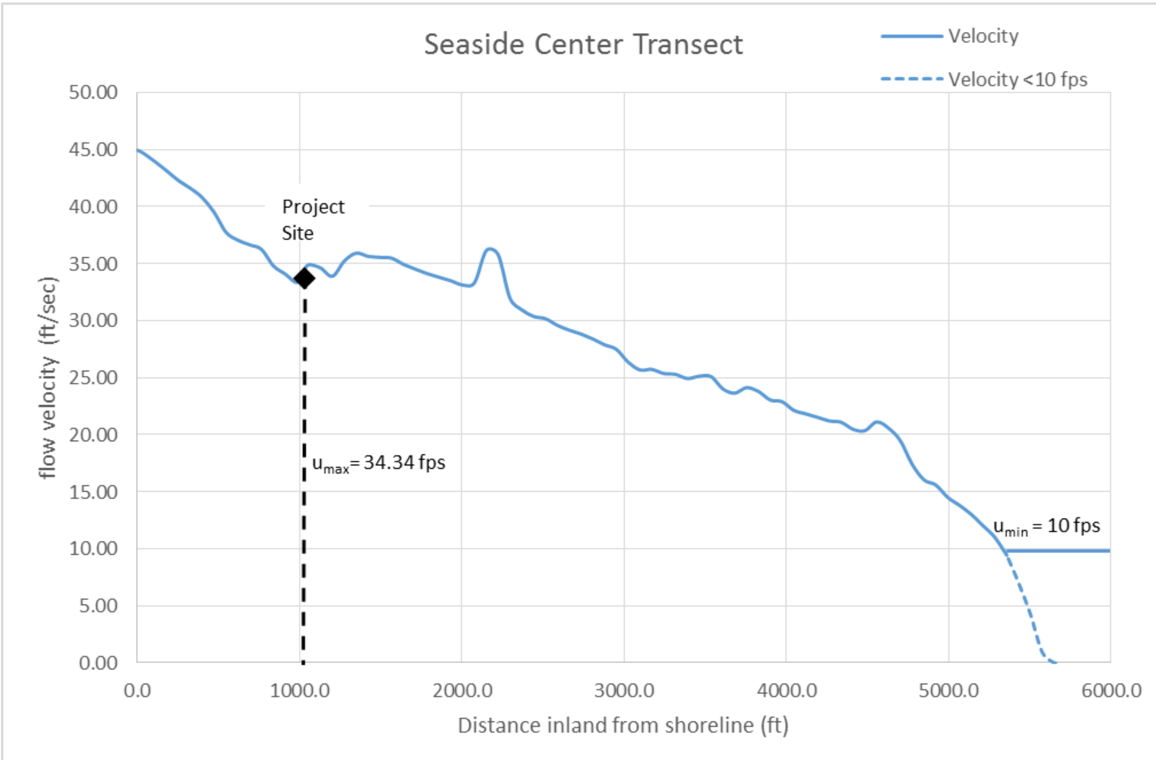

Figure A-12: Flow velocity (u_i) profile from Energy Grade Line analysis for Center transect

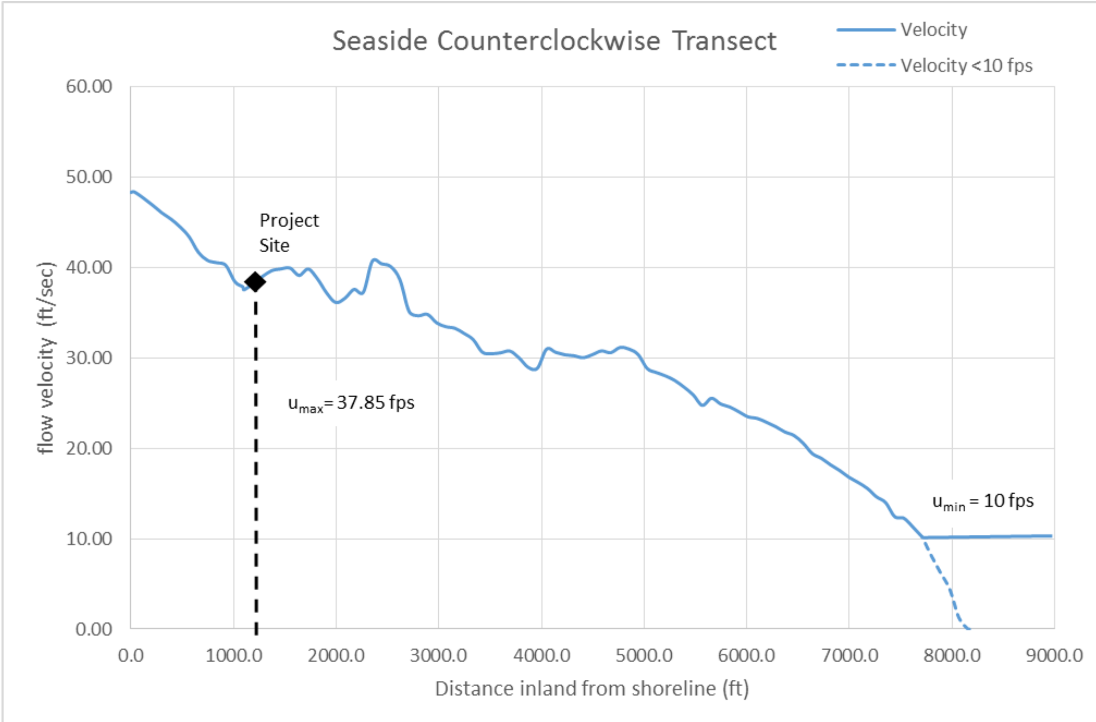

Figure A-13: Flow velocity (u_i) profile from Energy Grade Line analysis for Counterclockwise transect

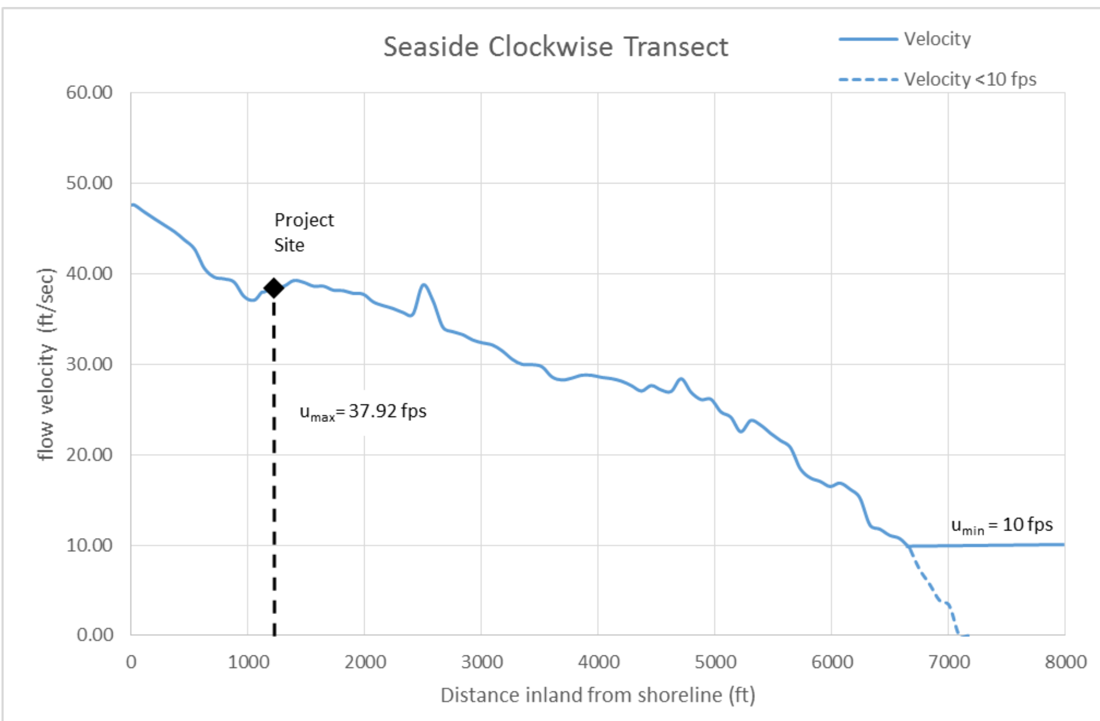

Figure A-14: Flow velocity (u_i) profile from Energy Grade Line analysis for Clockwise transect

All of the flow depths and flow velocities determined from the EGLA are listed in **Table A-2**

Table A-2: Results of Energy Grade Line Analysis for three transects through Monterey project site.

Transect	Maximum Flow Depth, h_{max} (ft)	Maximum Flow Velocity, u_{max} (ft/sec)
Center	26.65	34.34
Counterclockwise	30.44	37.85
Clockwise	**31.4**	**37.92**

A.6 Prototype Concrete Buildings

A.6.1 6-Story Office Building

The 6-story office building consists of a Special Moment Resisting Frame on the perimeter and selected interior frames, and interior gravity columns supporting posttensioned floor slabs (See **Figure A-15**). The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: D

Response Spectrum Parameters: S_s = 1.513, S_1 = 0.554, S_{DS} = 1.009, S_{D1} = 0.554

Structural System Response Factors: R = 8, Ω_o = 3, C_d = 5.5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 74 ft. With a maximum flow depth of 31.4 ft, this building could function as a "Refuge of Last Resort" at the 4th level (38 ft) up to the roof (if acceptable).

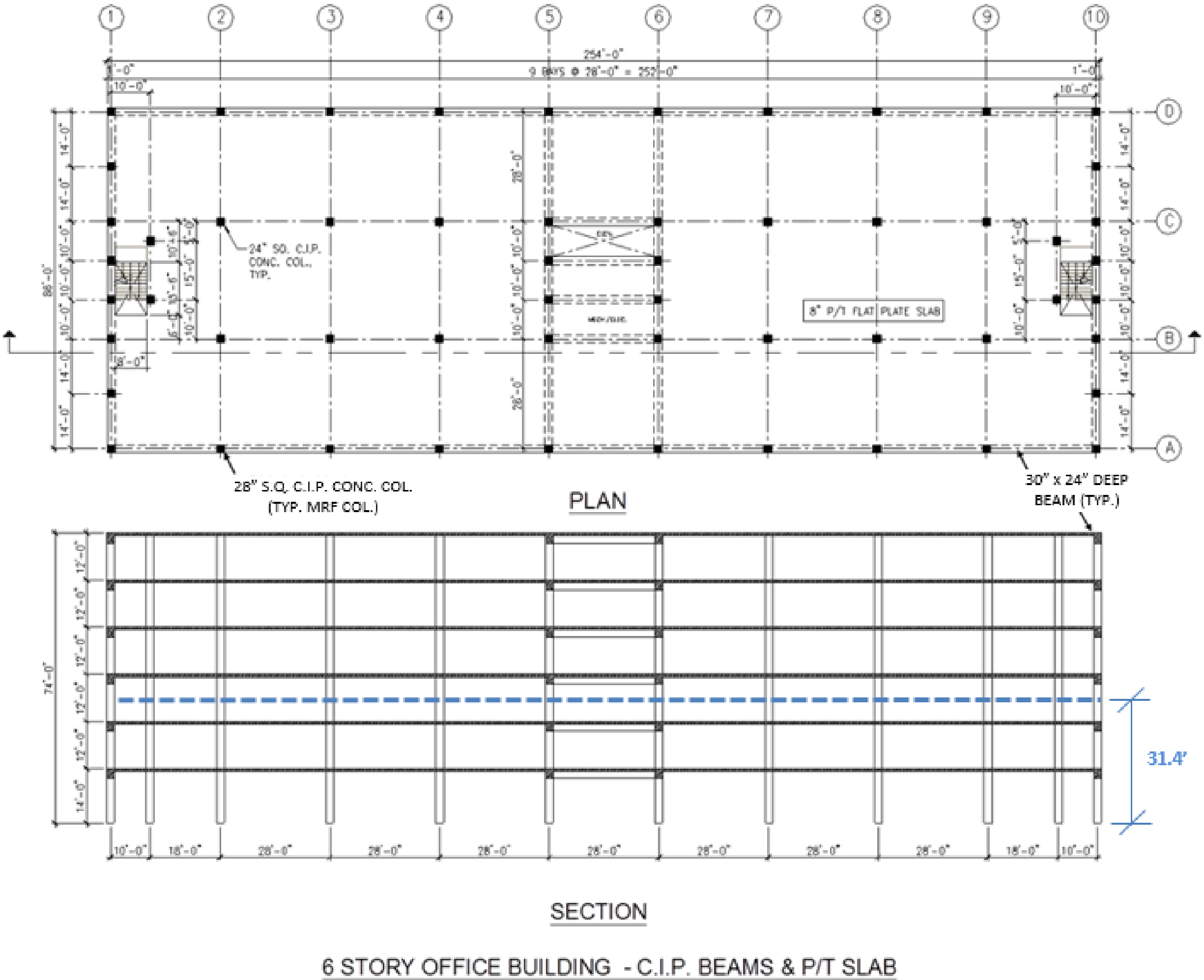

Figure A-15: 6 Story Office Building using Special Reinforced Concrete Moment Frame and posttensioned flat slab supported on gravity columns

A.6.2 7-Story Residential Building

The 7-story residential building consists of a Building Frame System with special reinforced concrete shear walls at exit stairs and elevator core, and interior gravity columns supporting posttensioned floor slabs (See **Figure A-16**). The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: D

Response Spectrum Parameters: S_s = 1.513, S_1 = 0.554, S_{DS} = 1.009, S_{D1} = 0.554

Structural System Response Factors: R = 6, Ω_o = 2.5, C_d = 5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 66 ft. With a maximum flow depth of 31.4 ft, this building could function as a “Refuge of Last Resort” at the 5th level (39 ft) up to the roof (if acceptable).

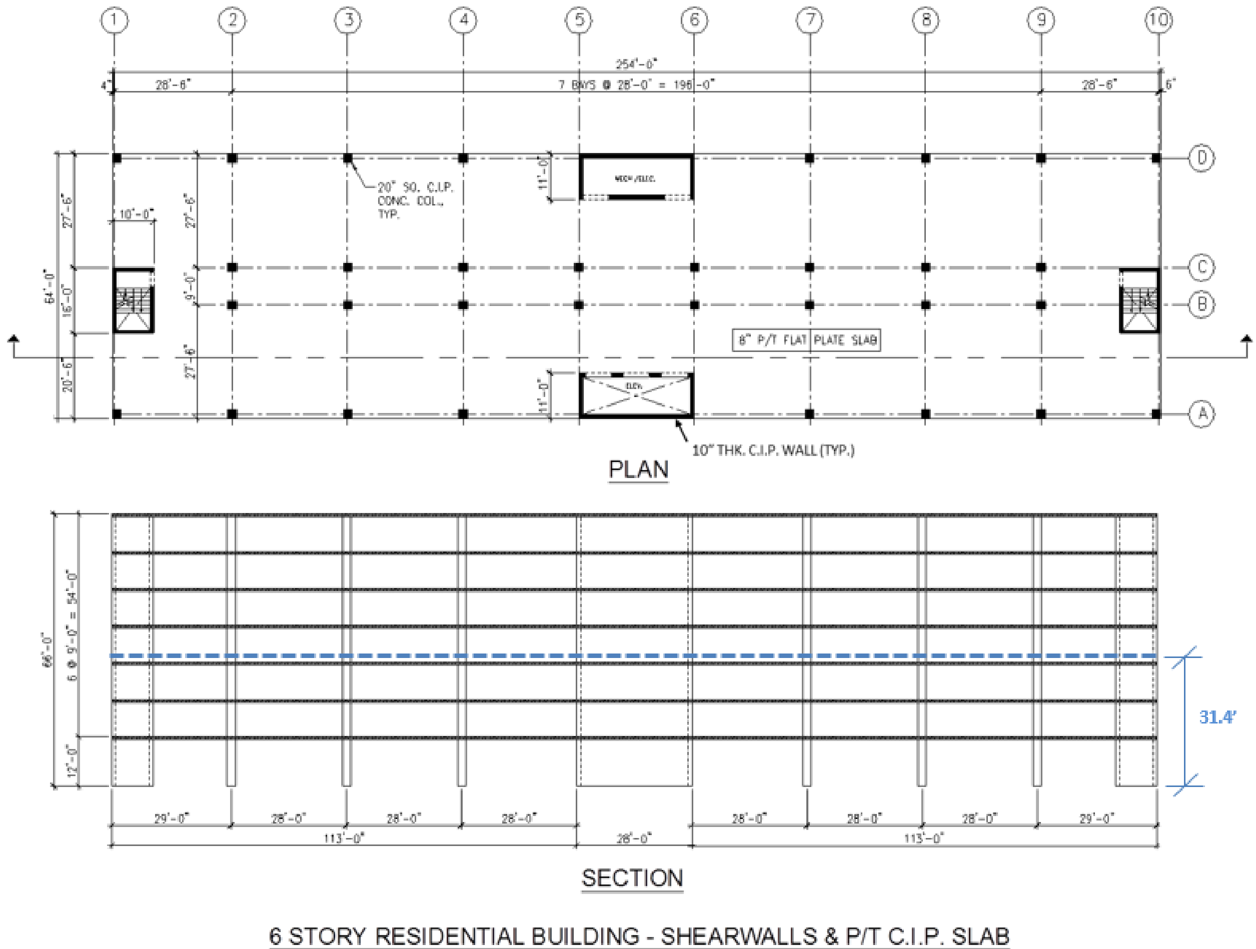

Figure A-16: 7 Story Residential Building using Special Reinforced Concrete Shear Walls and posttensioned flat slab supported on gravity columns

A.7 Tsunami Loading Summary

Table A-3 gives a summary of the tsunami loads determined for the building located at the selected site. The subsequent sections show the derivation of each of these values.

This section of this example shows detailed calculation of the tsunami loads, along with evaluation of the structural system and components for these loads. Note that these calculations are far more detailed than would be necessary for a typical design project because the intent here is to provide a complete explanation of the various calculations and their application.

Table A-3: Summary of Tsunami Loading for Office and Residential Buildings

Flow Parameters	Office Building	Residential Building
Max. Inundation Depth, h_{max} (ft)	31.4	31.4
Max. Flow Velocity, u_{max} (fps)	37.92	37.92
Overall Building Lateral Loading (kips)		
Load Case 1	2,295	2,295
Load Case 2	7,369	7,369
Load Case 3	1,226	1,226
Component Loading (kips)		
Exterior Column Hydrodynamic Drag	1,298 [2]	1,298 [2]
Interior Column Hydrodynamic Drag	132.4	110.4
Exterior Column Debris Impact	107.25 [3]	107.25 [3]
Exterior Wall Debris Impact	-	107.25 [3]
Wall and Slab Loading (psf)		
Hydrodynamic Pressure on Walls	-	3,163
Stagnation Pressure in Mech/Elec Rm	-	1,582 [5]
Surge Uplift on Elevated Slabs	-	20

[1] Including effect of debris damming, C_{cx}, applied to column tributary width.

[2] Limited by log crushing capacity.

[3] Stagnation pressure acting outwards on structural walls and floor slab enclosing Mech/Elec room corresponding to the maximum velocity and corresponding flow depth.

A.8 Assumed Conditions

The following conditions are assumed to apply for this example:

1. The building is oriented with the longitudinal axis parallel to the shoreline.
2. The building has no basement.
3. The foundation system consists of deep piles with pile caps supporting all shear walls and all exterior columns.
4. The ground floor slab-on-grade has isolation joints at all columns, structural walls and grade beams.
5. The top of the first floor windows is 8 feet above grade, with the window sill at 3 ft.
6. The building location is not in the vicinity of a shipping container storage yard or port facility, and is therefore not subject to debris impact from shipping containers, ships or barges.
7. The non-structural exterior cladding spans vertically between floors.

A.9 Tsunami Design for Office Building

A.9.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

In lieu of performing detailed hydrostatic and hydrodynamic analysis, **Eqn. 6.10.1-1** provides a simplified but conservative estimate of the maximum lateral load on the building.

$$\mathrm{f_{uw}} = 2.5\mathrm{I_{tsu}}\gamma_s \mathrm{h_{max}^2} = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 31.4^2 = 173\ \mathrm{kip/ft}$$

Assuming $C_{cx} = 0.7$ controls per **Section 6.8.7**

Then $\mathrm{F} = 0.7 \times 254 \times 173 = 30{,}854\ \mathrm{kips}$

This lateral load can be compared with 0.75 Ω_o E_h = 0.75 x 3 x 2,435 = 5,479 kips < 30,854 kips. The detailed analysis for LC2 and LC3 must therefore be performed as shown below. Overall Building Forces

A.9.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

A.9.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} $\leq h_{max}$ = 31.4 ft

$\leq$ 14 ft (ground floor story height)

$\leq$ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

For the sake of illustration, if we had assumed that the ground floor consists of structural grade beams and integral slab on grade without isolation joints, and that the soil allowed ground water pressure increase below the building (ie. sandy or gravely subsoil), the buoyancy would need to be considered as follows:

Section 6.9.1, Eqn. 6.9.1-1 $F_v = \gamma_s V_w$ = (1.1x64.0)(254' x 88' x 8')/1000 = 12,588 kips

Apply load combination: 0.9*D* + F_{TSU} + 1.2 H_{TSU}

where H_{TSU} = 0 *since scour is assumed uniform around the building perimeter*.

and building dead weight, D = 16,000 kips, *including foundation*.

Therefore net uplift = - 0.9 x 16,000 + 12,588 = -1812 kips, downward.

Overall uplift would therefore not be a concern, even if the ground floor were a structural slab capable of resisting the associated buoyancy pressures. This example also ignores any uplift resistance provided by the deep foundations.

In combination with buoyancy, Load Case 1 requires application of the associated hydrodynamic drag on the entire building.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4575 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254' overall width of building

h = 8'

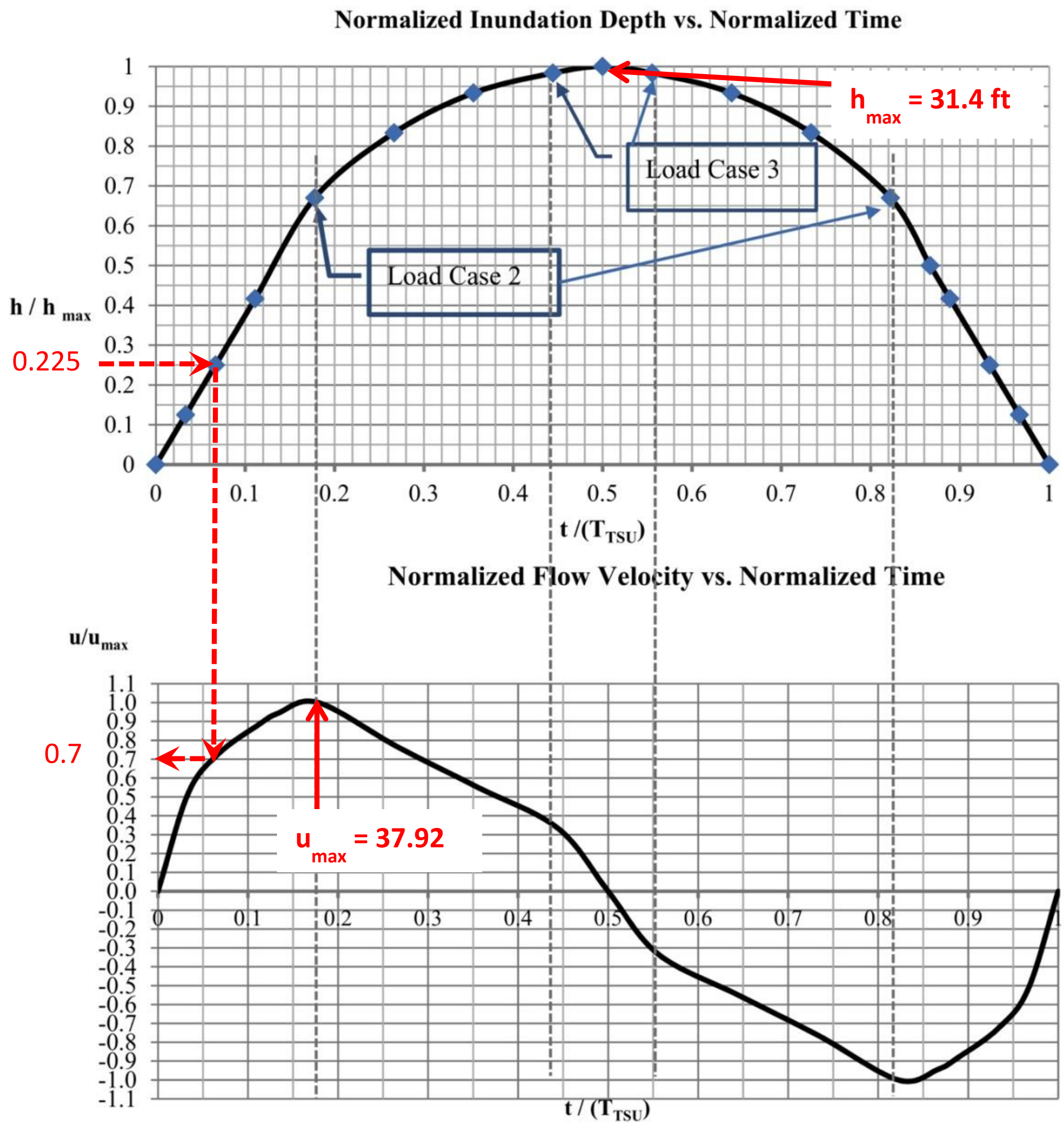

Figure A-17: Determining "u" for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/31.4 = 0.255. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.068. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.7. Therefore the flow velocity is u = 0.7 x 37.92 = 26.54 fps.

So$F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 26.54^2)/1000 = 2{,}295\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure at the

first floor level would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail so as to relieve lateral load on the structural frame. Note that only portion of this load will go to the second floor slab, which therefore has to be resisted by the lateral force resisting system. The majority of the load will go directly to the grade beam/foundation system. The entire lateral load must be resisted by the deep foundation assuming maximum scour has already occurred.

A.9.2.2 Load Case 2: Maximum Flow Velocity

In this particular example, LC1 and LC2 are very similar for the overall building, but the following calculation is shown for completeness.

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is 2/3h_{max} = 2/3 x 31.4 = 20.93 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.252 (**Table 6.10-1** based on B/h_{sx} = 254/20.93 = 12.14)

Since the inundation depth of 20.93 feet exceeds the bottom of the second floor beams (14' – 24"/12) = 12', the inundated area of the beams must be included in the closure coefficient, which is given by:

h_{sx} = 20.93 ft.

$h_{col\ EQ}$ = 20.93'-24"= 18.93' (Clear height of submerged Moment Resisting Frame columns)

$A_{col\ EQ}$= 18.93'x2.33'x40 = 1,764 ft^2 (40 MRF earthquake columns each 28" wide)

$h_{col\ Grv}$= 20.93'-8"= 20.26' (clear height of submerged gravity load columns)

$A_{col\ Grv}$= 20.26'x2'x16 = 648 ft^2 (16 gravity load column, each 2' wide)

A_{wall} = 0 ft^2 (no walls in MRF structure)

A_{beam} = 24"x254'x1= 508 ft^2 (1x24" deep beam goes above 4th level beam)

$$C_{cx} = \frac{\sum(A_{col} + A_{wall}) + 1.5A_{beam}}{Bh_{sx}} = \frac{\sum((1764 + 648) + 0) + 1.5 \times 508}{254' \times 20.93'} = 0.597 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

$u = u_{max}$ = 37.92 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.252 \times 0.7 \times 254(20.93 \times 37.92^2)/1000 = 7{,}369\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal or inland elevation of the building over a height of 20.93 feet above grade as shown in **Figure A-18**. The lateral force resisting system for the structure at the first and second floor levels would be evaluated for this load.

A.9.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 31.4 ft. and the flow velocity is 1/3u_{max} = 1/3 x 37.92 = 12.64 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/31.4 = 8.1)

Since the inundation depth of 31.4 feet exceeds the bottom of the third floor beams (14' + 12'– 24") = 24', the inundated area of the beams must be included in the second through third floor closure coefficient, which is given by:

h_{sx} = 31.4 ft.

$h_{col\ EQ}$ = 31.4'-24"-24"= 27.4'

$A_{col\ EQ}$= 27.4'x2.33'x40 = 2,557 ft^2

$h_{col\ Grv}$= 31.4'-8"-8"= 30.07'

$A_{col\ Grv}$= 30.07'x2'x16 = 962 ft^2

A_{wall} = 0 ft^2

A_{beam} = 24"x254'x2= 1016 ft^2

$$C_{cx} = \frac{\Sigma(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\Sigma((2557+962)+0)+1.5\times1016}{254\prime\times31.4\prime} = 0.632 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 12.64 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(31.4 \times 12.64^2)/1000 =$ $1{,}226\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 31.4 feet above grade as shown in **Figure A-19**. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth for hydrodynamic load debris impact.

A.9.3 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category D, **Section 6.8.3.4** permits the use of $0.75\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear. From the seismic design of this structure, E_h = 2,435 kips. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 3 \times 2{,}435 = 5{,}479\ kips$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 7,369 kips applied over a height of 20.93 ft. A portion of this load will be resisted by the grade beam/foundation system as shown in **Figure A-18**, reducing the overall load by 2,464 kips. A portion of this load will be resisted by the grade beam/foundation system as shown in **Figure A-18** , reducing the overall load by 2,464 kips. Therefore, V_{TSU} = 7,368 – 2,464 = 4,904 kips. Applying the LFRS assessment gives:

$$0.75\Omega_o E_h = 5{,}479\ kips > 4{,}904\ kips \quad \therefore OK$$

So the lateral force resisting system has the capacity to resist the overall tsunami loads. In order to combine these systemic effects with the individual component loads on each member of the lateral force resisting system, the building must be analyzed for a seismic base shear, E_h, of:

$$E_h = \frac{V_{TSU}}{0.75\Omega_o} = \frac{4{,}904}{0.75 \times 3} = 2{,}180\ kips$$

This seismic base shear must be distributed up the height of the building following ASCE 7 seismic design provisions. This Load Case 2 base shear of 2,180 kips was applied to the same ETABS model used for the original wind and seismic analysis of the building. Load Case 3 was also analyzed but did not control any of the member designs. This ETABS analysis resulted in the column forces shown in **Figure A-21** to **Figure A-23** for floors one through three, respectively. These systemic loads on each element of the LFRS must be combined with the component loads on that member.

While acting as part of the lateral force resisting system, these columns are also subjected to component drag or debris impact loads. According to ASCE 7 Section 6.8.3.5, the columns in the inundated floors must be designed and detailed for these higher forces "*that result from the overall tsunami forces on the structural system combined with any resultant actions caused by the tsunami pressures acting locally on the individual structural components for that direction of flow*". All members of the LFRS must resist the forces resulting from the overall system analysis, in combination with hydrodynamic and impact loads acting on the member itself.

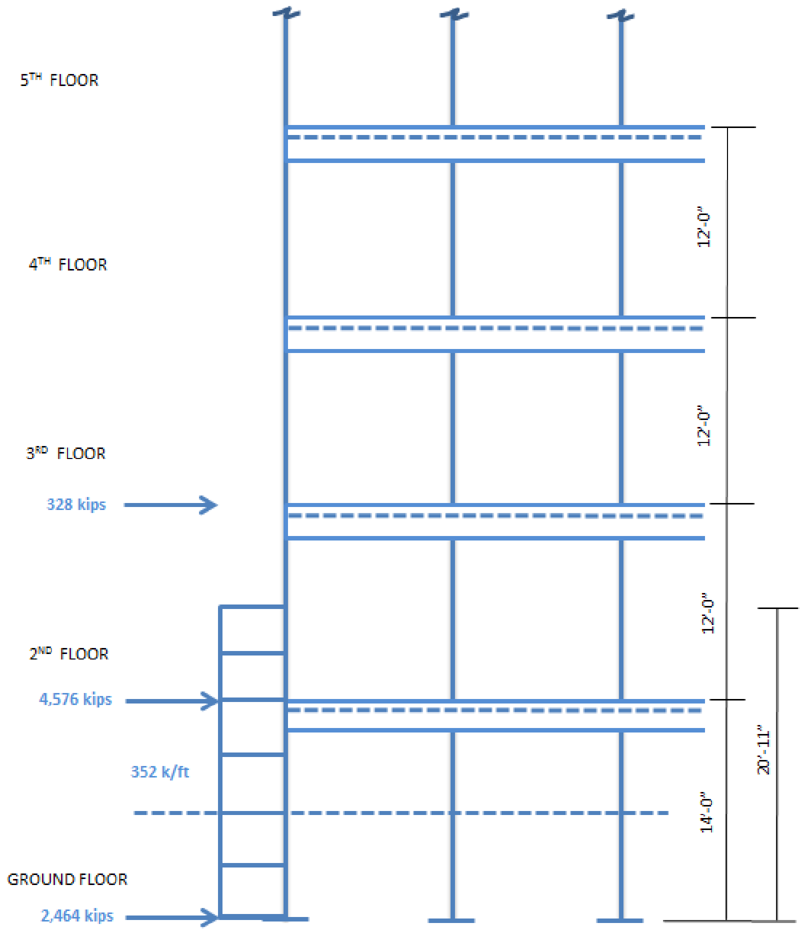

Figure A-18: LC2 Tsunami loads on overall Seaside office building

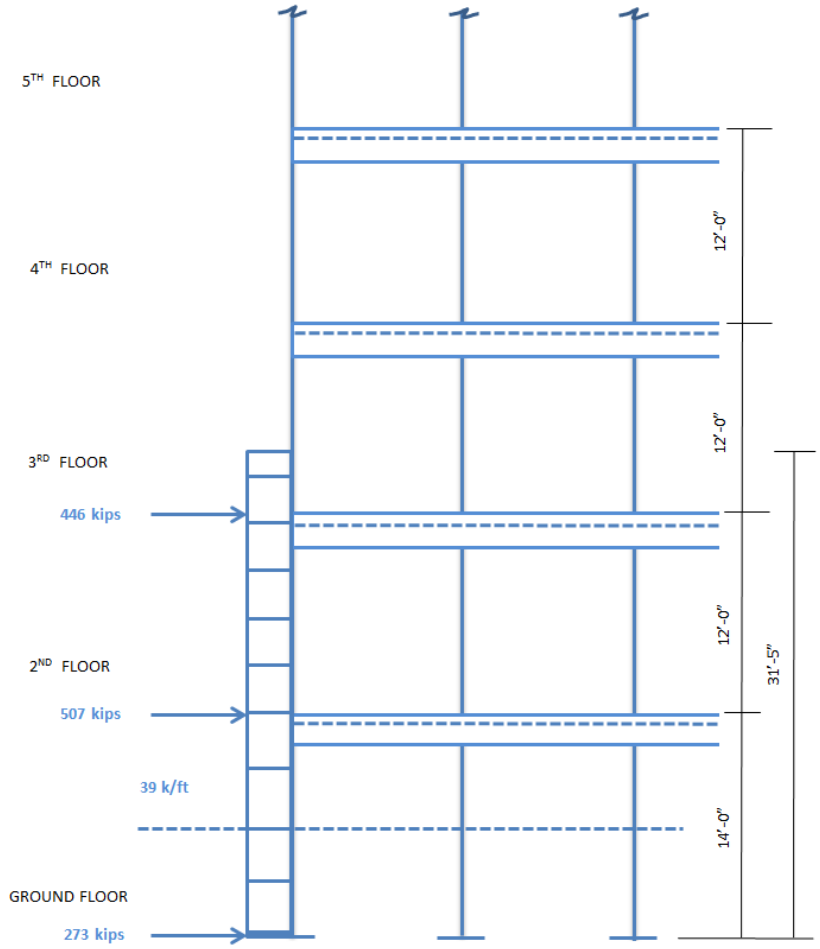

Figure A-19: LC3 Tsunami loads on overall Seaside office building

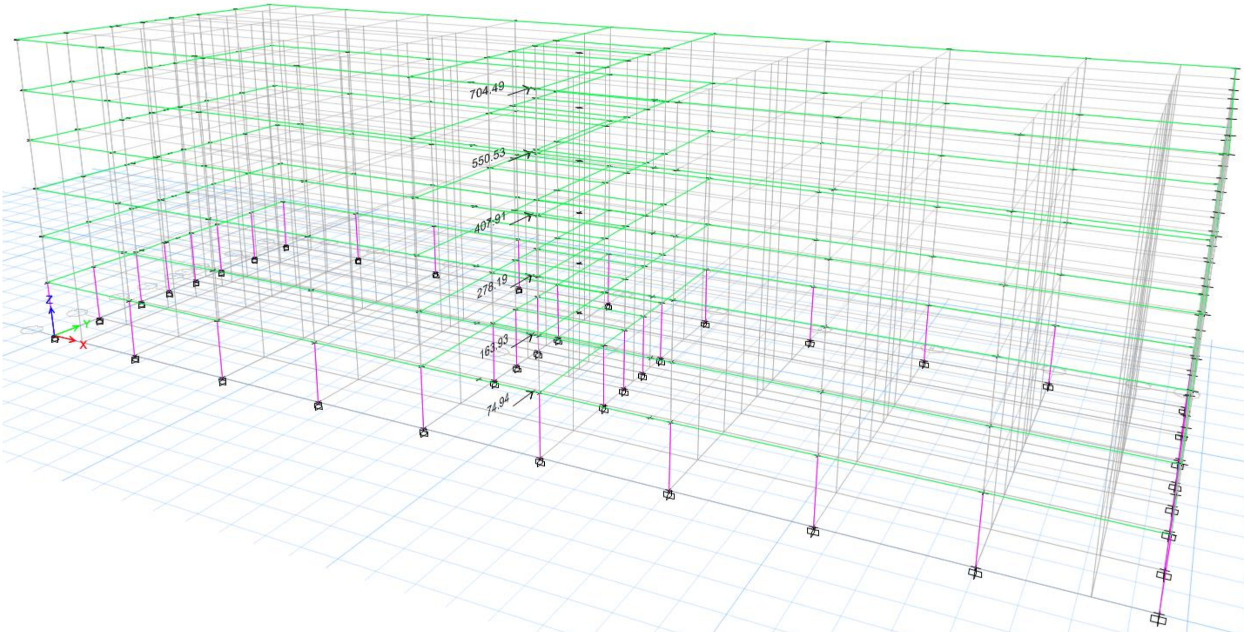

Figure A-20: ETABS computer model of Moment Resisting Frame office building (with seismic lateral loads shown).

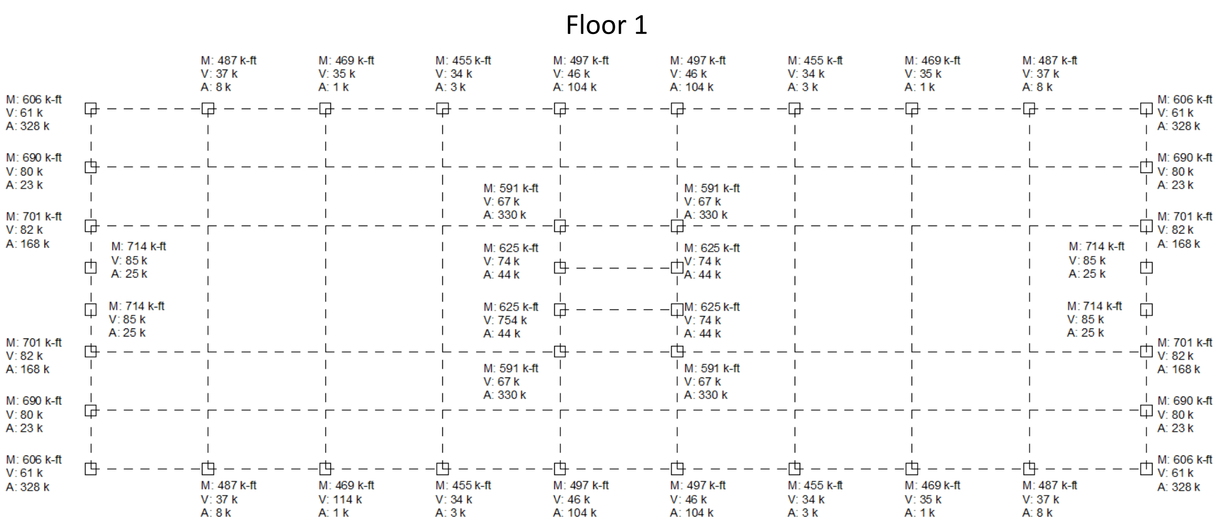

Figure A-21: Maximum forces in the first floor columns due to tsunami base shear

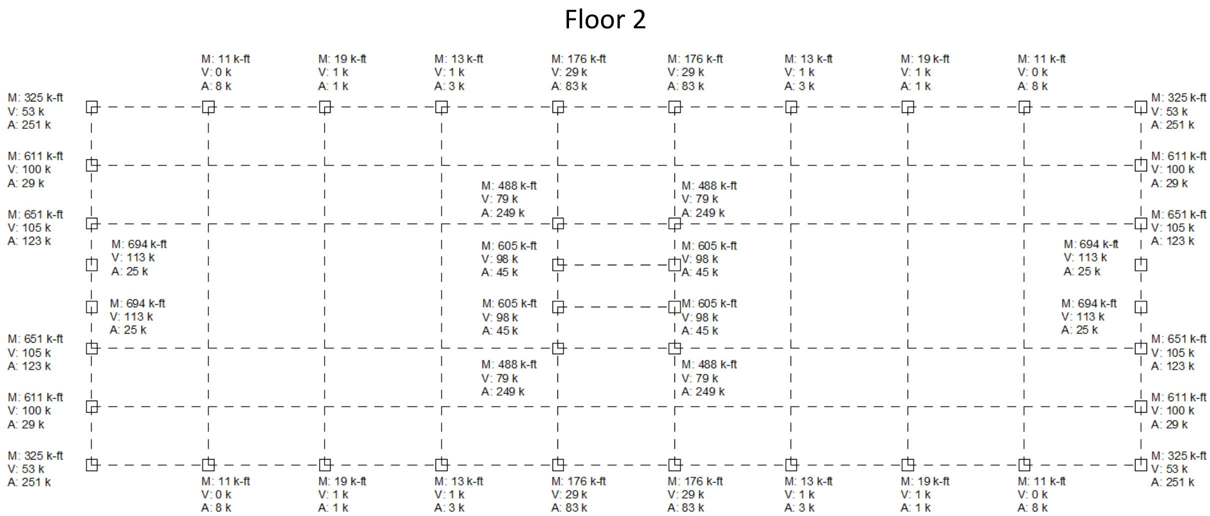

Figure A-22: Maximum forces in the second floor columns due to tsunami base shear

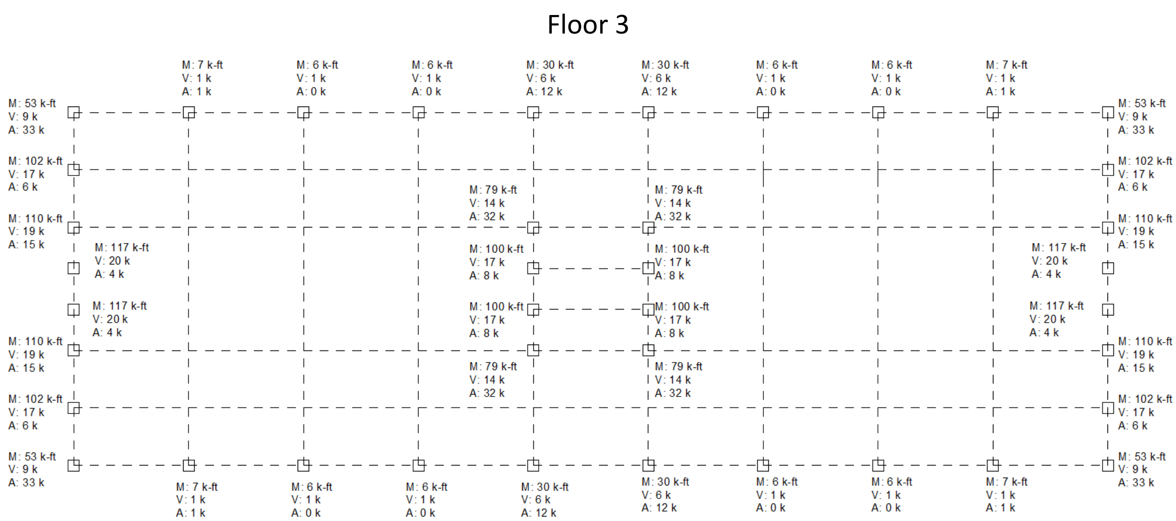

Figure A-23: Maximum forces in the third floor columns due to tsunami base shear

A.10 Component Design

A.10.1 Drag Force on Components - Section 6.10.2.2

A.10.1.1 Exterior Columns

For Load Case 1, the exterior cladding is assumed to remain intact. Since the cladding spans vertically between floors for this example building, none of the hydrodynamic lateral load in LC1 will be applied directly to the ground floor columns. [*Note that if the exterior cladding were supported by girts which transferred lateral load to the columns, then the columns would need to be designed for this load*.]

For Load Cases 2 and 3, the exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** will require that C_d = 2.0 and the width dimension, *b*, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Previous calculation of C_{cx} showed that the default value of 0.7 controls for LC2 and LC3 for this building, Therefore *b* = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 20.93 ft and u_{max} = 37.92 fps.

The hydrodynamic drag is computed using **Eqn 6.10-4** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(20.93 \times 37.92^2)/1000 = 1{,}298\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 1,298/20.93 = 62.0 kips/ft over the lower 20.93 feet of the column. The column must be designed for this load combined with gravity loads using the load combinations in **Section 6.8.3.3**. In addition, because the exterior columns are part of the LFRS, these component loads must be combined with the systemic forces and the column designed for the combined loads.

A.10.1.2 Interior Columns

Interior columns are 24" (2 ft) square R.C. columns. For Load Case 1, the interior is not yet inundated, so there are no hydrodynamic loads on the interior columns. The controlling load case will be LC2, when the inundation depth is h_e = 20.93 ft and u_{max} = 37.92 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**)

Therefore $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 2.0(20.93 \times 37.92^2)/1000 = 132\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 132/20.93 = 6.3 kips/ft over the lower 20.93 feet of the column. This load must be combined with gravity loads using the load combinations in **Section 6.8.3.3** and the column capacity verified.

A.10.2 Other Hydrodynamic Loads

No other hydrodynamic load conditions apply to this building since there are no structural walls and the spandrel beam is integral with the slab so the lateral load on the beam will transfers directly to the slab diaphragm.

A.10.3 Debris Impact Loads - Section 6.11

The inundation depth at the site exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads per **Section 6.11**.

A.10.3.1 Detailed Debris Impact Calculation for Office Building

Wood Logs and Poles - Section 6.11.2

The nominal maximum instantaneous debris impact force is given by **Eqn. 6.11-2** as:

$$F_{ni} = u_{max}\sqrt{km_d}$$

Where u_{max} = 37.92 fps

$k = EA/L$ for the wood log with a minimum value of 350 k/in ($4.2\text{x}10^6$ lb/ft)

m_d = 1000/32.2 = 31.1 slugs for the minimum 1000 lb log.

Therefore: $$F_{ni} = u_{max}\sqrt{km_d} = 37.92\sqrt{4.2 \times 10^6 \times 31.1}/1000 = 433\ kips$$

The design instantaneous debris impact force is then given by **Eqn. 6.11-3** as:

$$F_i = I_{tsu}C_0F_{ni} = 1.0\ \times 0.65 \times 433 = 281\ kips$$

The impulse duration is given by **Eqn. 6.11-4** as:

$$t_d = \frac{2m_du_{max}}{F_{ni}} = \frac{2\times31.1\times37.92}{433{,}060} = \ 0.0054\ sec$$

The column can be designed using a dynamic analysis by applying an impulsive rectangular pulse with magnitude F_i and duration t_d. Alternatively an equivalent elastic static analysis can be performed of the column subjected to F_i multiplied by a dynamic response factor, R_{max}, given in **Table 6.11-1**. The ratio of impact duration to natural period of the impacted structural element is obtained using t_d and the natural period of the column assumed to be fixed-fixed. For this case, the natural period is given by;

$$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}}$$

Where L = unbraced column length = 14' – 24" = 12 ft for the ground floor columns.

ρ = column mass per unit length = 2.333'x2.333'x150pcf/32.2ft/s^2 = 25.36 slugs/ft

E = modulus of elasticity of the column concrete = 3600 ksi = $518.4\text{x}10^6$ psf

I = moment of inertia of column section = $bd^3/12$ = 2.333x2.333^3/12 = 2.47 ft^4

Therefore $$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}} = 2\pi\left[\frac{12^2}{22.373}\right]\sqrt{\frac{25.36}{518.4\times10^6\times2.47}} = 0.00444\ sec$$

The ratio of impact duration to column natural period is therefore t_d/T_{col} = 0.0054/0.00444 = 1.22.

Table 6.11-1 gives the dynamic response factor R_{max} = 1.6, therefore the equivalent static load is given by;

$$F_{es} = R_{max}F_i = 1.6 \times 281 = 450\ kips.$$

This exceeds the maximum required impact force of 107.25 kips, therefore the column can be evaluated for a lateral point load of 107.25 kips applied at locations which are critical for flexure and shear.

A.10.4 Impact by Vehicles – Section 6.11.3

The impact force is given as $F_i = I_{tsu}$ x 30 = 30 kips. This will not control over the log impact load determined above.

A.10.5 Impact by Submerged Tumbling Boulder and Concrete Debris – Section 6.11.4

Because h_{max} = 31.4 ft > 6 ft, an impact force of $F_i = I_{tsu}$ x 8 = 8 kips shall be applied at 2ft above grade. This will not control over the log impact load determined above.

A.10.5.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330C_0I_{tsu} = 330 \times .65 \times 1.0 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force will be reducible to 50%, or 107.25 kips. This load must be applied to the exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with other tsunami loads and it need not be applied to interior columns. . It is not combined with hydrodynamic loads on the column, but it must be combined with systemic loads if the member is part of the lateral force resisting system. In the event that this load exceeds the column capacity, a detailed debris impact analysis can be performed. Debris impact loads are not applied to interior columns.

In the event that this load exceeds the column capacity, a detailed debris impact dynamic analysis can be performed.

A.11 Column Design for Tsunami Loads

A.11.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 from **Figure A-15**. The column is part of the lateral force resisting system for longitudinal seismic load designed and detailed for Seismic Design Category D. The column has been designed for gravity and seismic loads resulting in the cross-section shown in **Figure A-24** and **Figure A-25** at the ground floor level and **Figure A-26** and **Figure A-27** for the remaining floor levels. The column will now be checked for tsunami load combinations.

Seismic design of the columns requires additional column ties to ensure ductility of the yield zones at each end of the column. These zones have a length equal to the maximum column cross-section dimension, in this case 28 inches. The critical shear force in this yielding zone occurs at a distance "*d*" from the top and bottom of the column, where *d* = 28 - 1.5 - 0.5 - 0.635 = 25.365 in. The critical shear force for the internal section of the column occurs at “d + h” from the edge of the column, where d + h = 25.365 + 28 = 53.365 in. The column ties required for seismic design will be evaluated for the shears induced by the tsunami both in the end section and center section of the column (**Figure A-28**).

Floor 1

End Section (A)

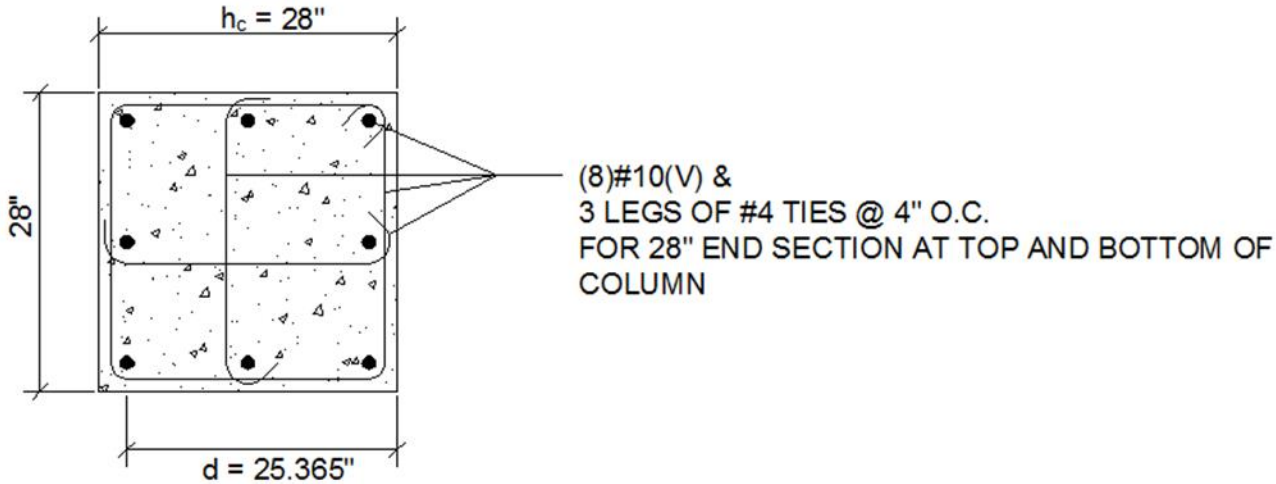

Figure A-24: Exterior column, cross-section at end of column at ground floor level based on SDC D design.

Center Section (B)

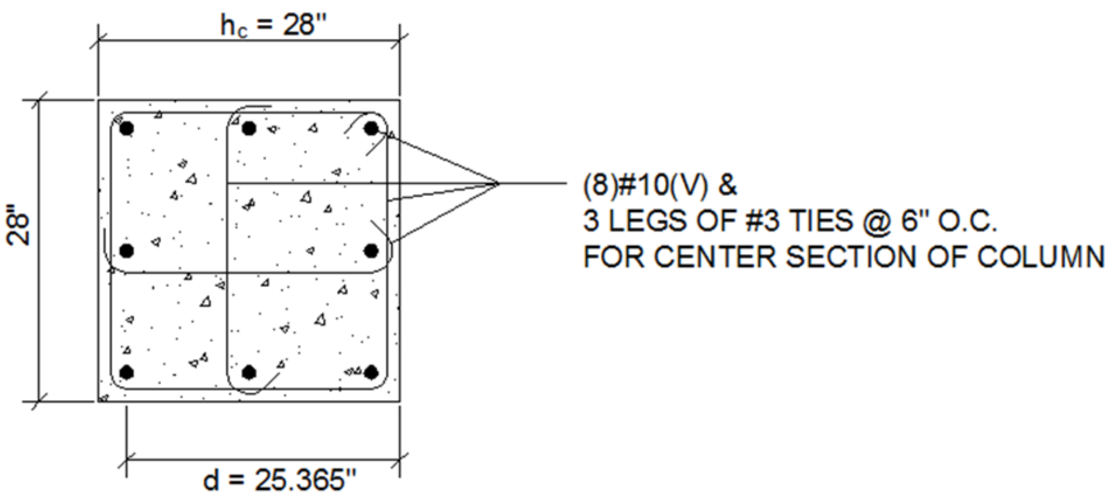

Figure A-25: Exterior column, cross-section at center of column at ground floor level based on SDC D design.

Floor 2 – 6

End Section (A)

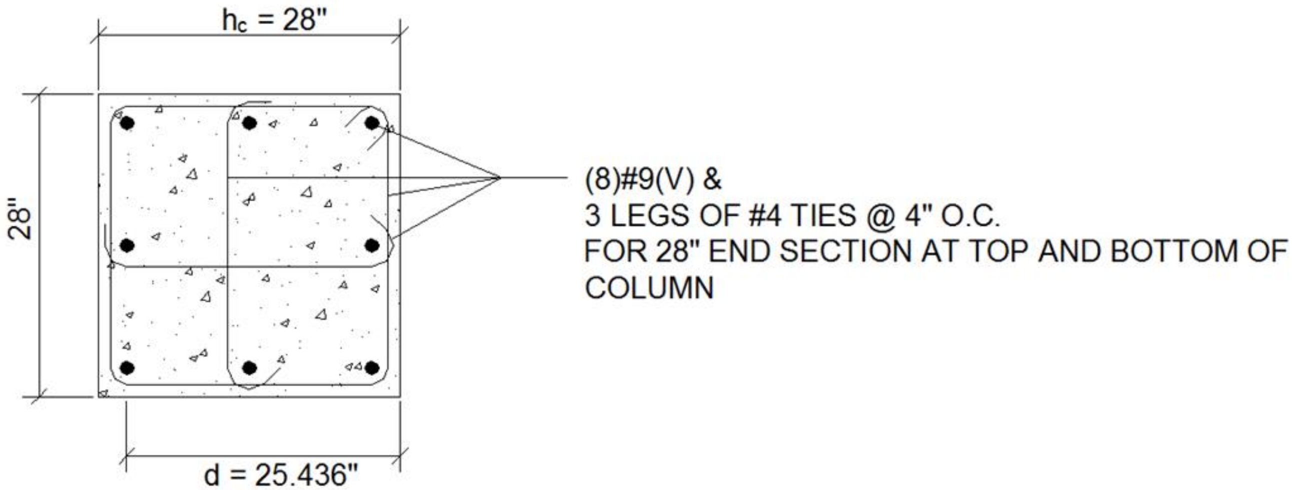

Figure A-26: Exterior column, cross-section at end of column at 2nd – 6th floor levels based on SDC D design.

Center Section (B)

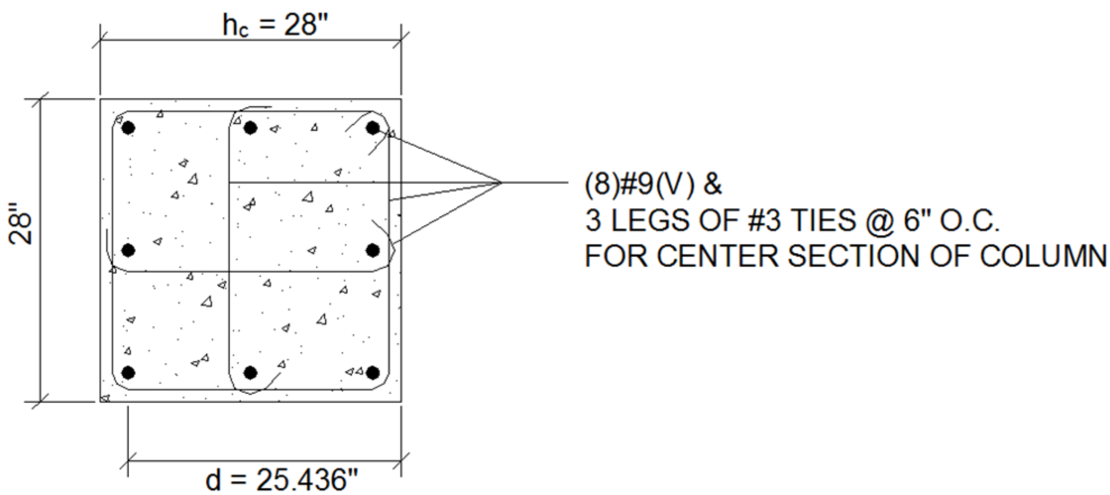

Figure A-27: Exterior column, cross-section at center of column at 2nd – 6th floor levels based on SDC D design.

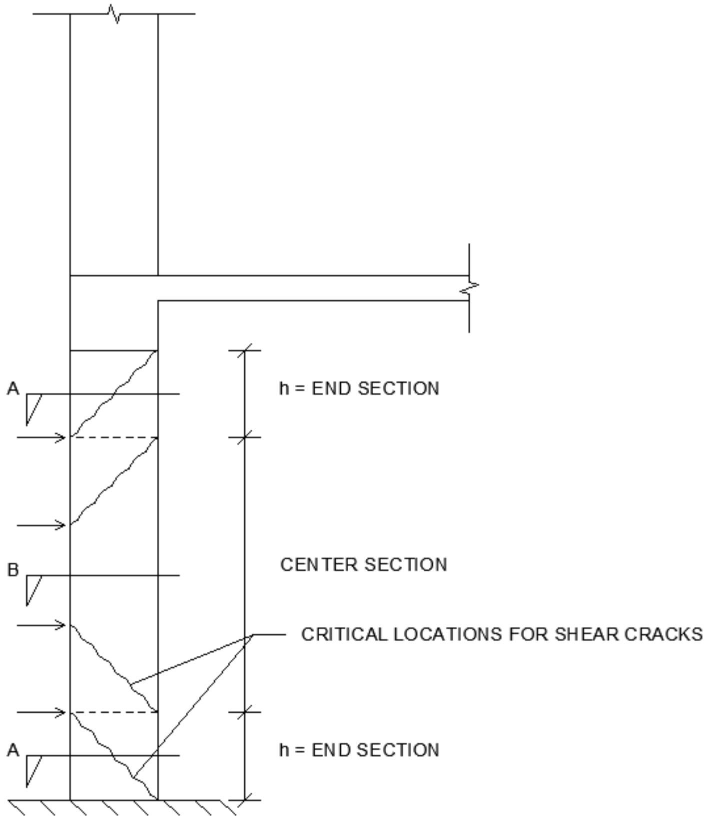

Figure A-28: Typical exterior column elevation showing end and center sections

A.11.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 15 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

$$P_D = [120(28)(15)(6)+(1.16)(2.5)(150)(28-2.5)+90(28)(5)+2.5^2(150)(74)]/1000 = 395\ k$$

Floor Live load reduction factor = $0.25+15/[4(15)(28)(5)]^{0.5} = 0.414$,

therefore, live load at the column base is: $P_L = 0.414[65(15)(28)(5)]/1000 = 56.5\ k$

Roof Live Load reduction factor = $R_1R_2 = [1.2-(0.001)(15)(28)](1.0) = 0.78$,

therefore, column roof live load is: $P_{Lr} = 0.78(20)(15)(28)/1000 = 6.55k$

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

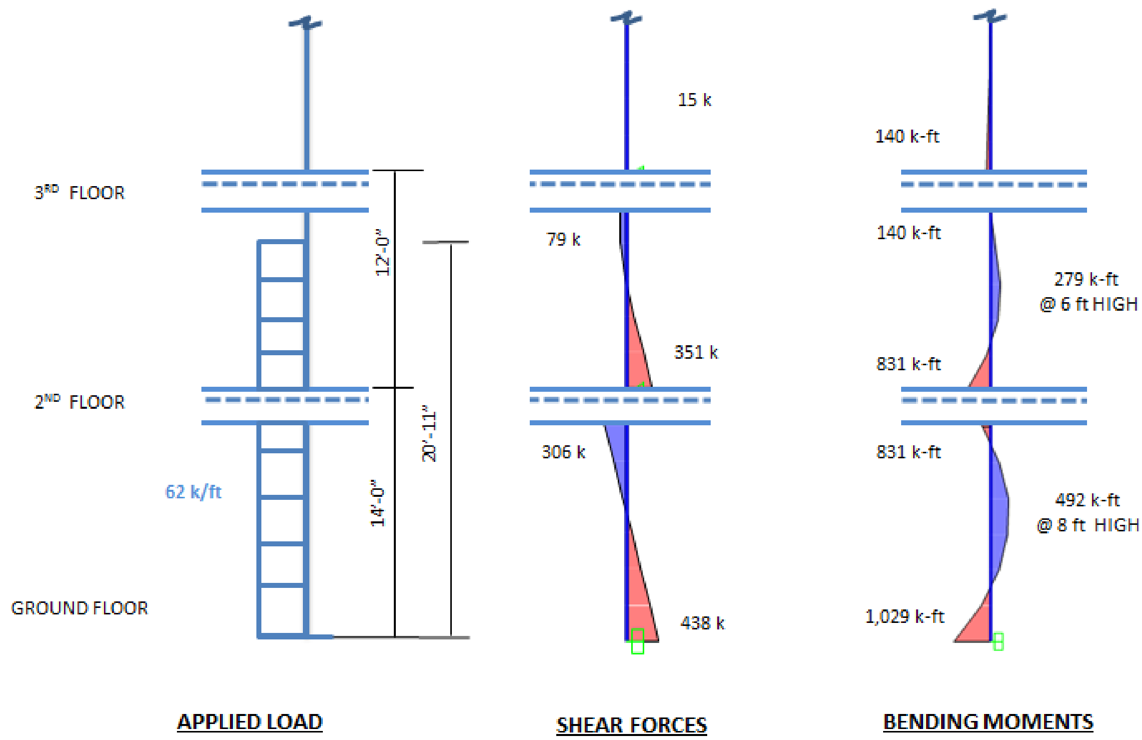

Figure A-29: Hydrodynamic loading on exterior column of the Seaside office building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just below the beam at each inundated floor for the maximum shear in the end section of the column. A log strike is also assumed to act just outside the end section (at "d + h_c") and at the mid-height of the clear column height for the maximum shear force and bending moment in the center section, respectively. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure A-30** to **Figure A-32**. The resulting shear force and bending moment diagrams for log impact at a distance "d + h_c" from the end of the column at each floor level are shown in **Figure A-33** to **Figure A-35**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown in **Figure A-36** to **Figure A-38**.

Impact load at d:

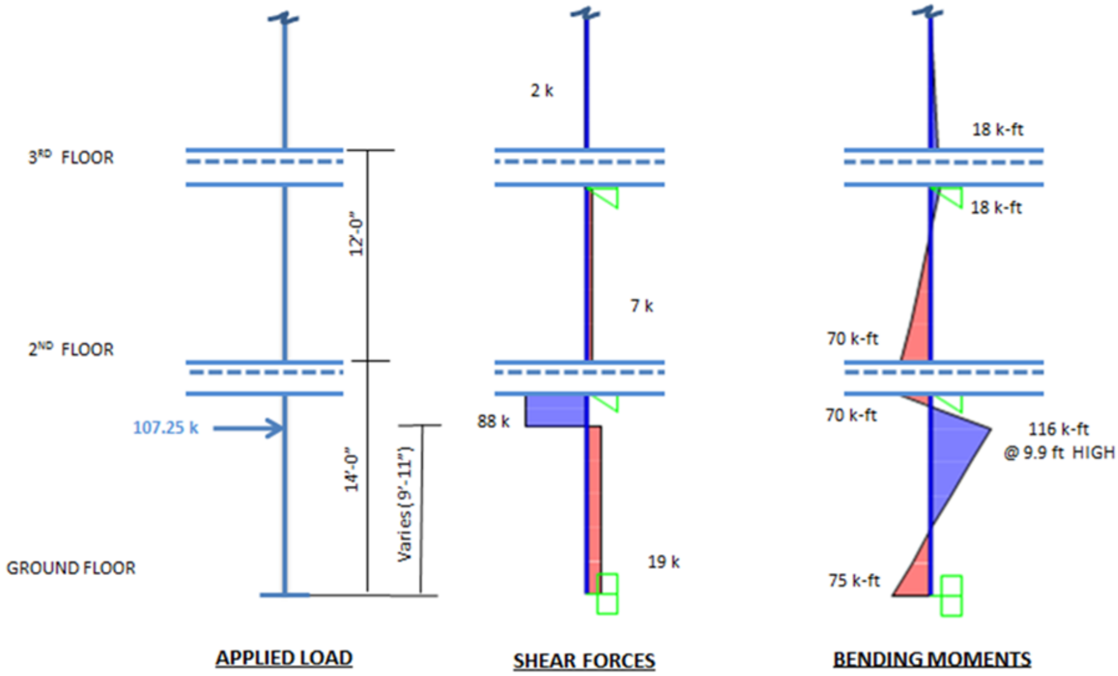

Figure A-30: Impact load applied at "d" away from the end of column on the ground floor

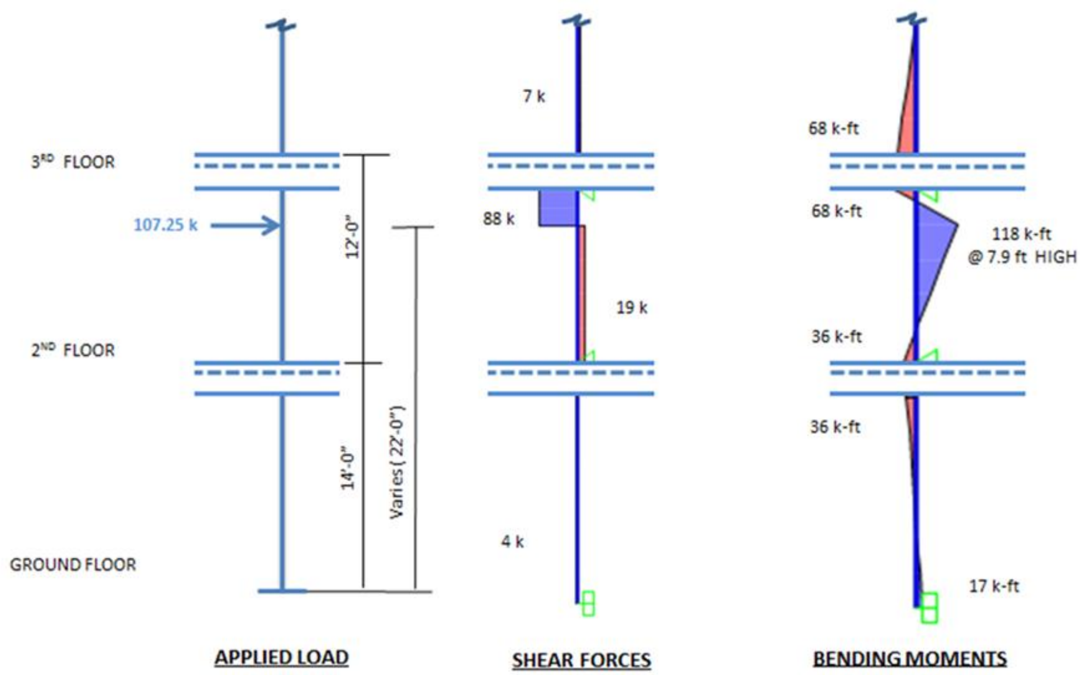

Figure A-31: Impact load applied at "d" away from the end of column on the 2nd floor

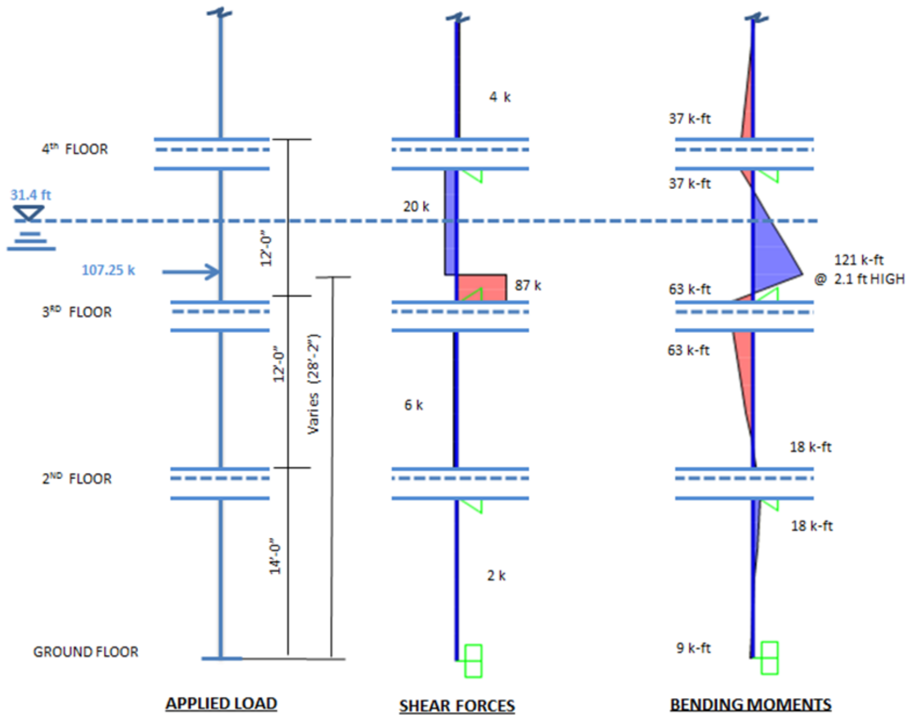

Figure A-32: Impact load applied at "d" away from the end of column on the 3rd floor

Impact load at d + h_c:

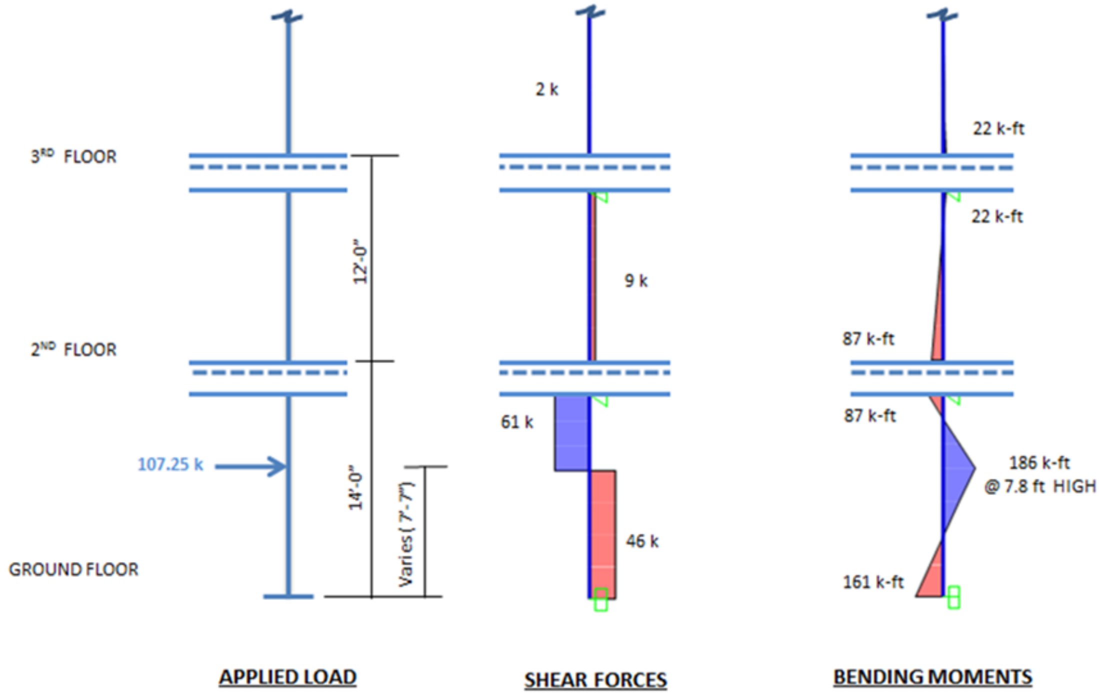

Figure A-33: Impact load applied at "d + h_c" away from the end of column on the ground floor

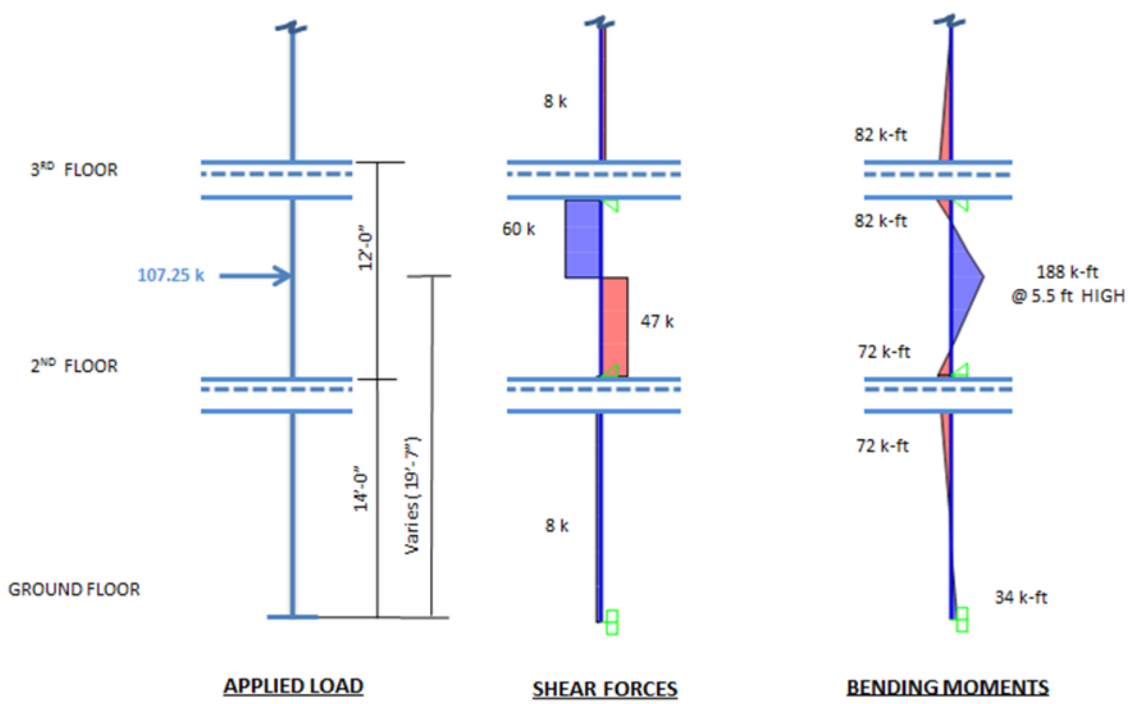

Figure A-34: Impact load applied at "d + h_c" away from the end of column on the 2nd floor

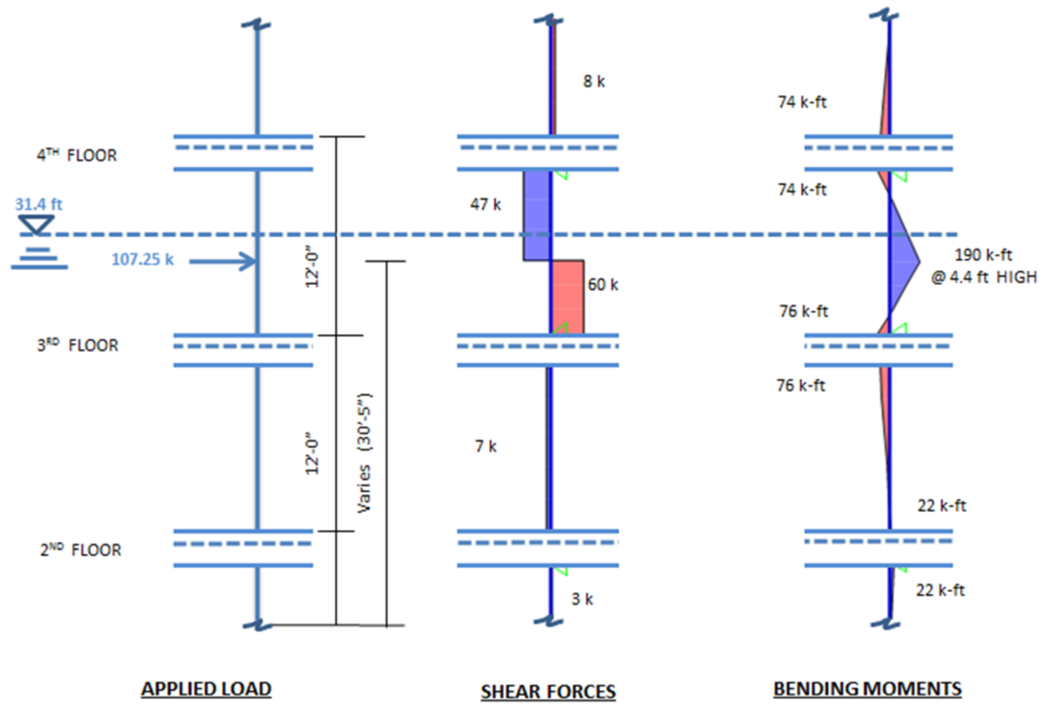

Figure A-35: Impact load applied at "d + h_c" away from the end of column on the 3rd floor

Impact load at mid-height:

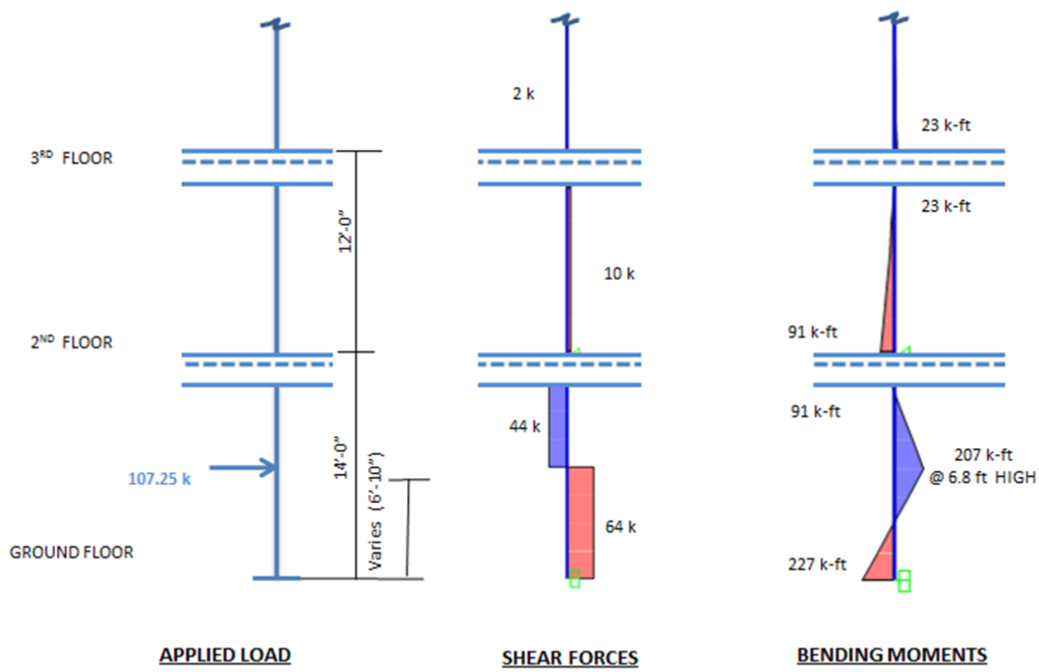

Figure A-36: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

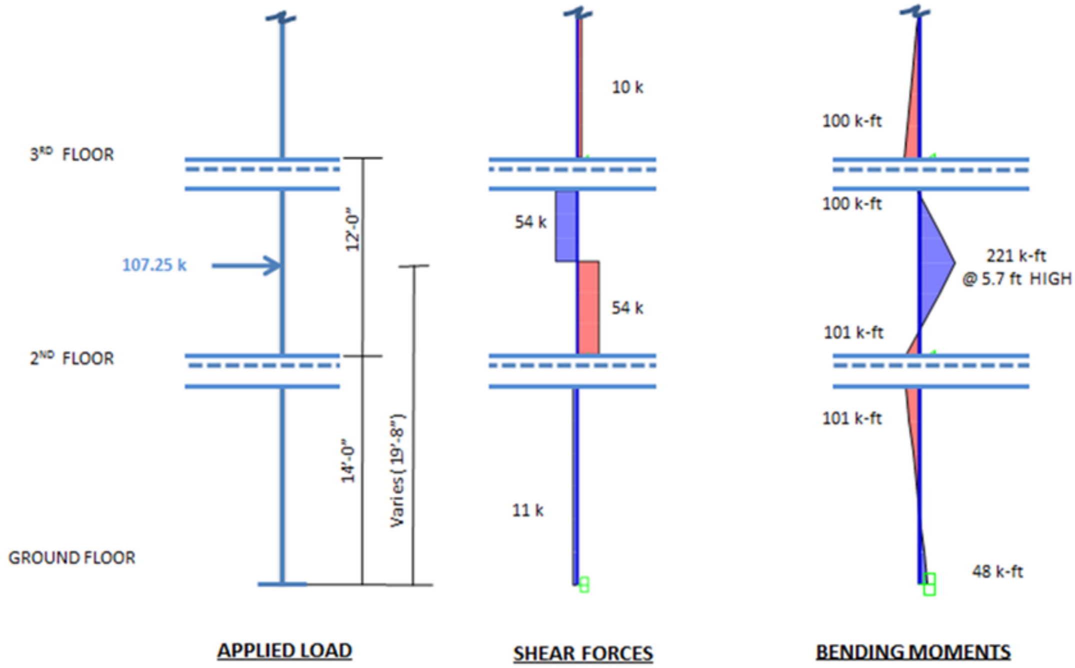

Figure A-37: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

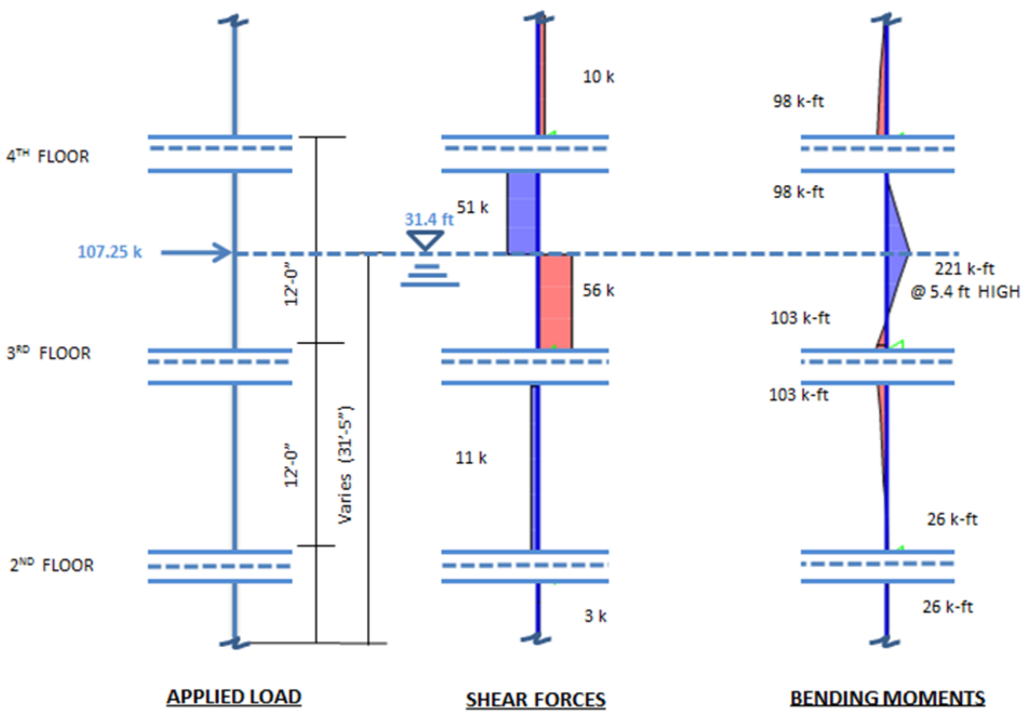

Figure A-38: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

Table A-4 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and log impact (Impact). In addition, because all of the exterior columns are part of the LFRS, **Table A-4**

also lists the maximum axial load, bending moment and shear forces determined by the ETABS analysis for the modified base shear (Overall) (See Section A.9.3). These “Overall” systemic forces are then combined with the controlling component forces (either “Hydro” or “Impact”) to obtain the “Combined” forces. Columns that are part of the transverse MRFs experience larger systemic loads and are therefore considered separately, along with columns having similar loads (“Special”).

The original column designs will now be evaluated for these load combinations and modified if necessary.

Table A-4: Results from loading conditions of Seaside office building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
1029	502.3	307	162	1.2D+Ftsu+0.5L (Hydro)
1029	355.5	307	162	0.9D+Ftsu (Hydro)
227	502.3	88	61	1.2D+Ftsu+0.5L (Impact)
227	355.5	88	61	0.9D+Ftsu (Impact)
701	334.25	85	85	1.2D+Ftsu+0.5L (Overall)
701	187.5	85	85	0.9D+Ftsu (Overall)
1526	398.25	392	247	1.2D+Ftsu+0.5L (Combined)
1526	251.5	392	247	0.9D+Ftsu (Combined)
Floor 2				
831	418.5	220	75	1.2D+Ftsu+0.5L (Hydro)
831	296.3	220	75	0.9D+Ftsu (Hydro)
221	418.5	88	60	1.2D+Ftsu+0.5L (Impact)
221	296.3	88	60	0.9D+Ftsu (Impact)
651	295.5	113	113	1.2D+Ftsu+0.5L (Overall)
651	173.25	113	113	0.9D+Ftsu (Overall)
1007	335.5	333	188	1.2D+Ftsu+0.5L (Combined)
1007	213.25	333	188	0.9D+Ftsu (Combined)
Floor 3				
140	334.8	15	15	1.2D+Ftsu+0.5L (Hydro)
140	237	15	15	0.9D+Ftsu (Hydro)
221	334.8	87	60	1.2D+Ftsu+0.5L (Impact)
221	237	87	60	0.9D+Ftsu (Impact)
117	330.8	20	20	1.2D+Ftsu+0.5L (Overall)
117	233	20	20	0.9D+Ftsu (Overall)
328	330.8	107	80	1.2D+Ftsu+0.5L (Combined)
328	233	107	80	0.9D+Ftsu (Combined)
Floor 4				
35	251.1	4	4	1.2D+Ftsu+0.5L (Hydro)
35	177.8	4	4	0.9D+Ftsu (Hydro)
98	251.1	10	10	1.2D+Ftsu+0.5L (Impact)
98	177.8	10	10	0.9D+Ftsu (Impact)
Floor 5				
9	167.4	1	1	1.2D+Ftsu+0.5L (Hydro)
9	118.5	1	1	0.9D+Ftsu (Hydro)
24	167.4	3	3	1.2D+Ftsu+0.5L (Impact)
24	118.5	3	3	0.9D+Ftsu (Impact)
Floor 6				
2	83.7	0	0	1.2D+Ftsu+0.5L (Hydro)
2	59.3	0	0	0.9D+Ftsu (Hydro)
6	83.7	1	1	1.2D+Ftsu+0.5L (Impact)
6	59.3	1	1	0.9D+Ftsu (Impact)

A.11.1.2 <u>Existing Exterior Column Design for Combined Gravity and Tsunami Loads</u>
The column chosen at Grid Intersection A-3 from **Figure A-15** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure A-39 to **Figure A-43** shows the interaction diagram for the typical exterior column including the tsunami load combinations.

The blue solid line (Original Column Design Strength) represents the design strength for the original columns. The green dashed line (New Column Design Strength) represents the design strength needed if one were to take into account only the hydrodynamic and impact loads shown in **Figure A-29** to **Figure A-38**. The dotted red line (New Overall Column Design Strength) represents the design strength needed for taking into account only the overall building forces for each column shown in **Figure A-21** to **Figure A-23**. . The orange dot-dashed line (New Combined Column Design Strength) represents the design strength needed for the overall loading combined with the hydrodynamic and impact loads per column. This series of plots is shown in alternating figures from **Figure A-39** to **Figure A-43** for all affected floor levels**.** Alternating **Figure A-40** to **Figure A-44** show the interaction diagrams for the combined forces with the controlling load combination for each column.

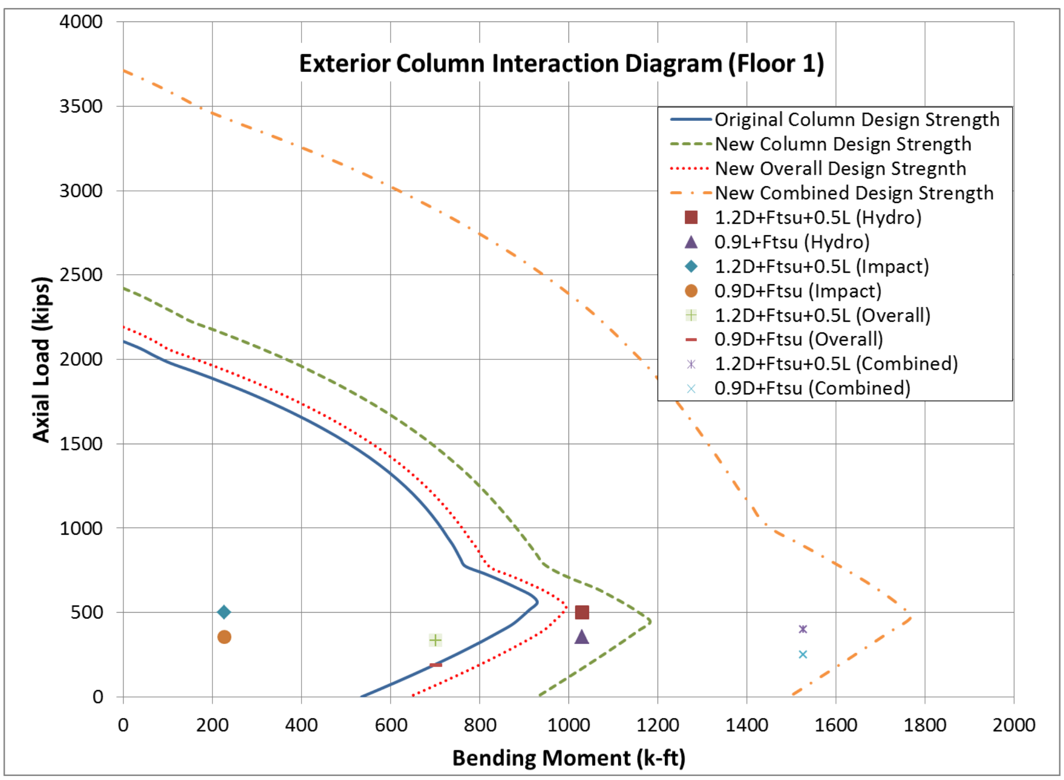

Figure A-39: Sequence of interaction diagrams for typical ground floor exterior column showing tsunami load combinations

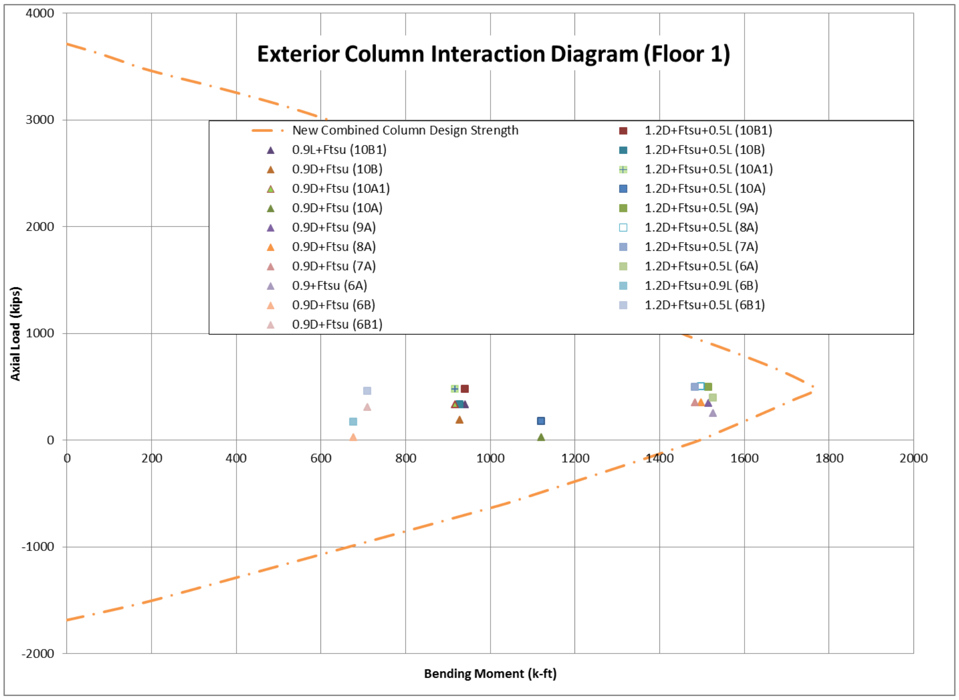

Figure A-40: Interaction diagrams for typical ground floor exterior column showing all combined tsunami load combinations

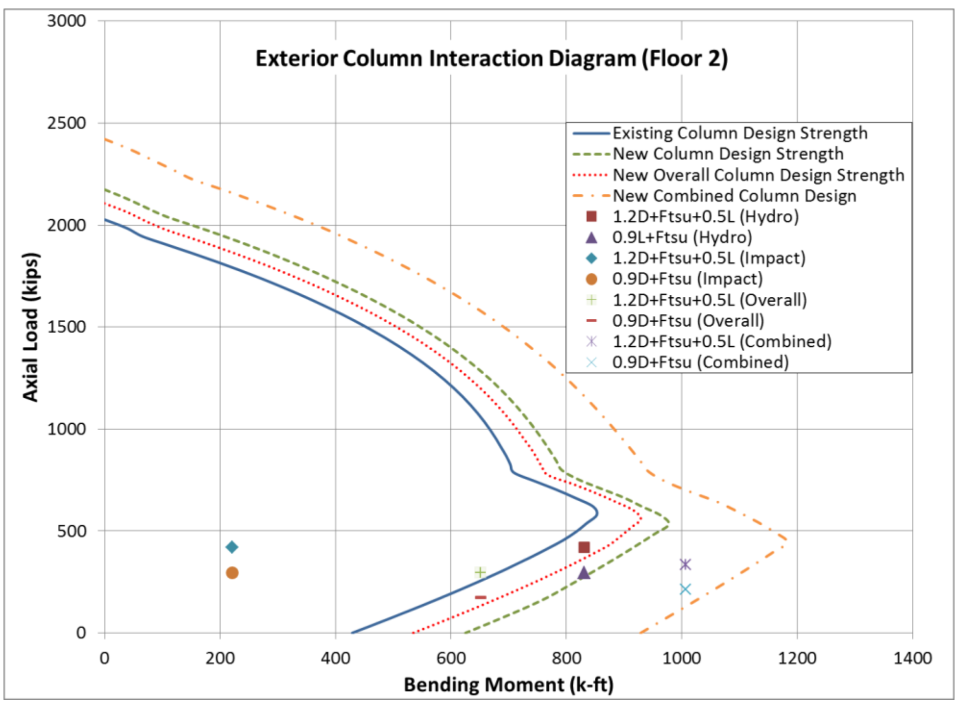

Figure A-41: Sequence of interaction diagrams for typical 2nd floor exterior column showing tsunami load combinations

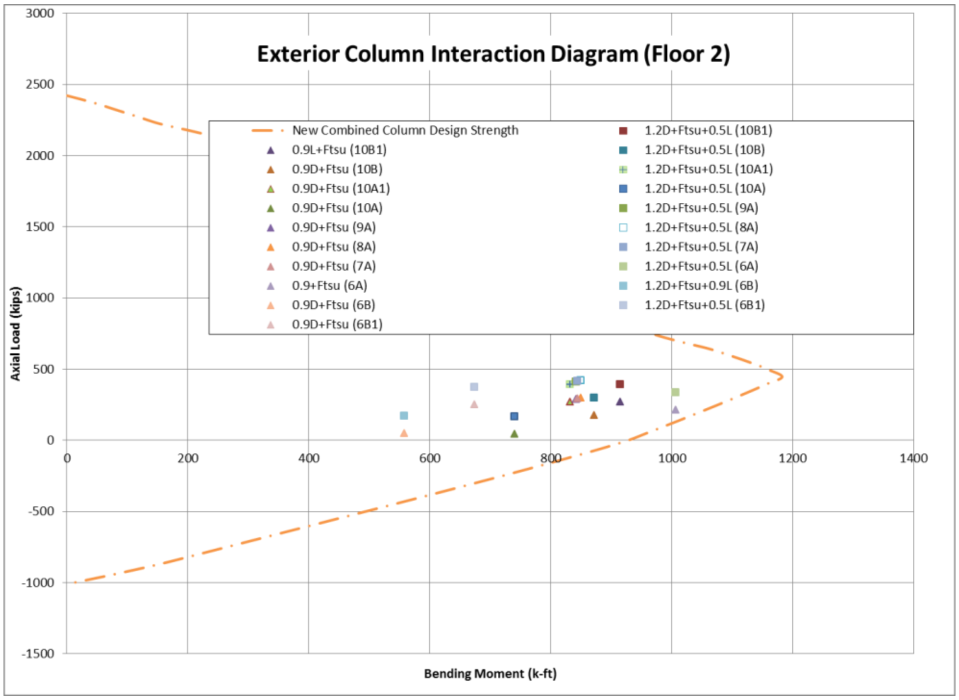

Figure A-42: Interaction diagrams for typical 2nd floor exterior column showing all combined tsunami load combinations

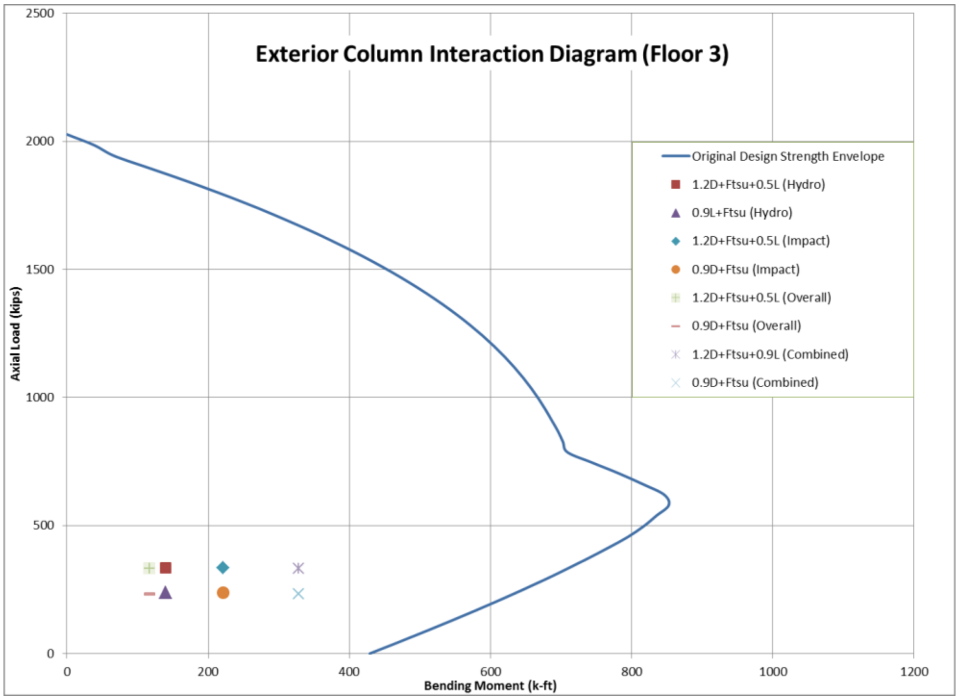

Figure A-43: Sequence of interaction diagrams for typical 3rd floor exterior column showing tsunami load combinations

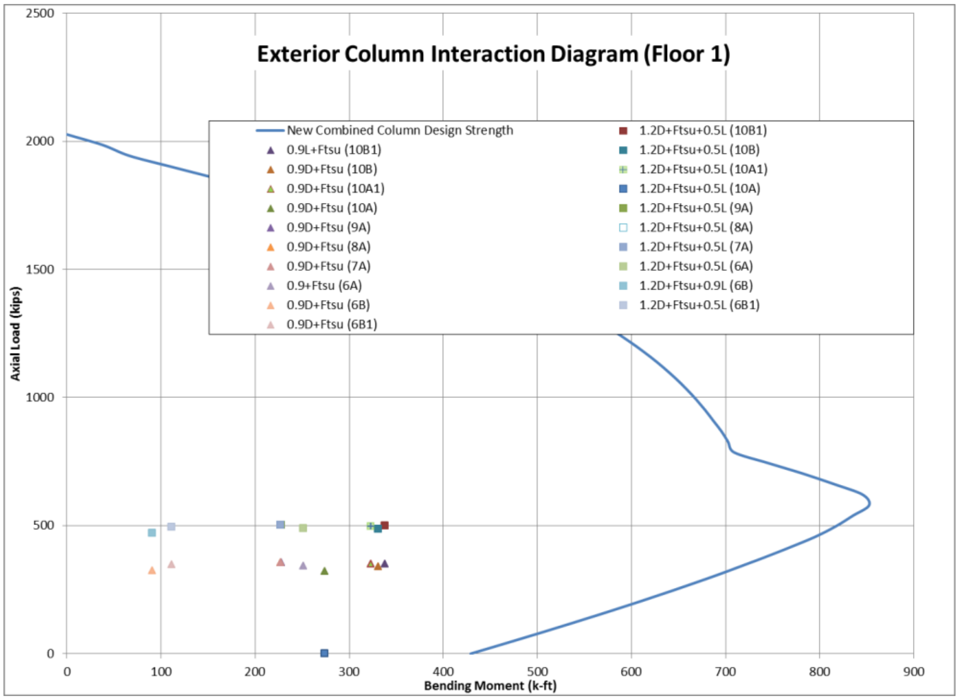

Figure A-44: Interaction diagrams for typical 3rd floor exterior column showing all combined tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

A.11.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure A-39** to **Figure A-43** the original exterior columns are adequate for log impact load, but the columns at the ground and 2nd floors must be strengthened to resist bending due to the hydrodynamic and overall system loads. Revised column designs shown in **Figure A-45** to **Figure A-48** were developed to satisfy the combined hydrodynamic and overall loads. The interaction diagrams for these new columns are shown in **Figure A-39** to **Figure A-41**. The ties in these columns are designed in Section A.11.1.4 for the applied tsunami shear forces.

Floor 1

End Section (A)

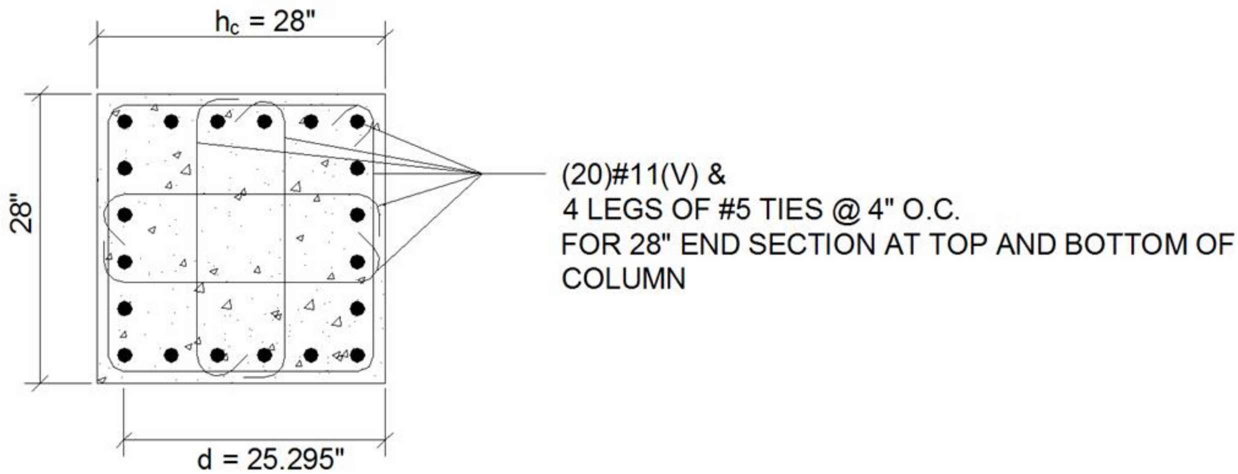

Figure A-45: Exterior column, cross-section at end section of column at ground floor level based on tsunami design requirements.

Center Section (B)

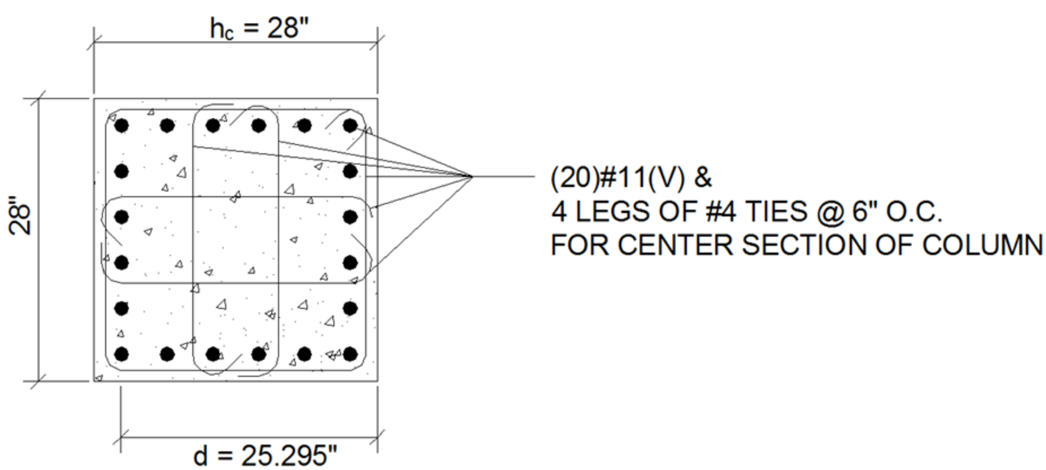

Figure A-46: Exterior column, cross-section at center section of column at ground floor level based on tsunami design requirements.

Floor 2

End Section (A)

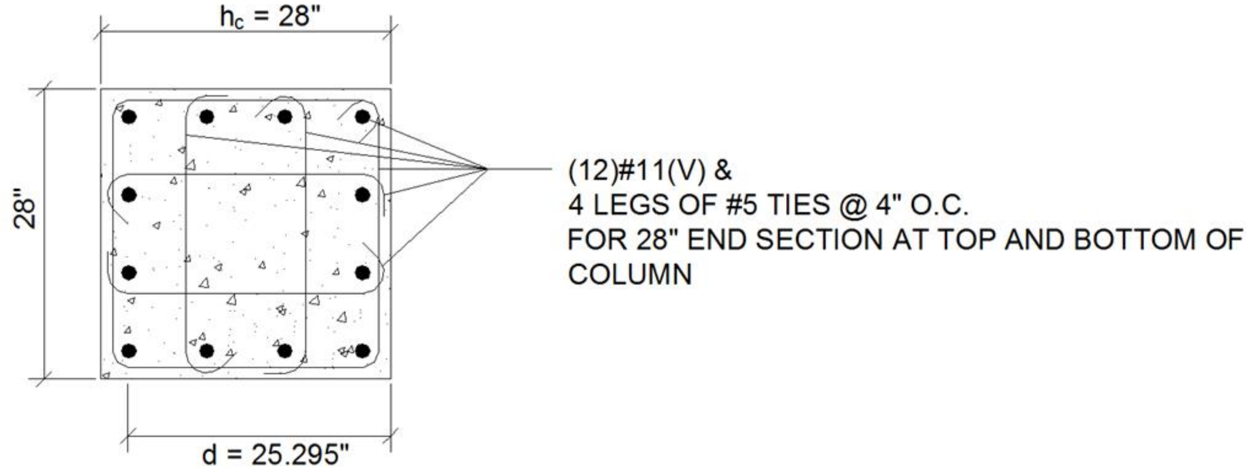

Figure A-47: Exterior column, cross section at end section of column at the 2nd floor level based on tsunami design requirements.

Center Section (B)

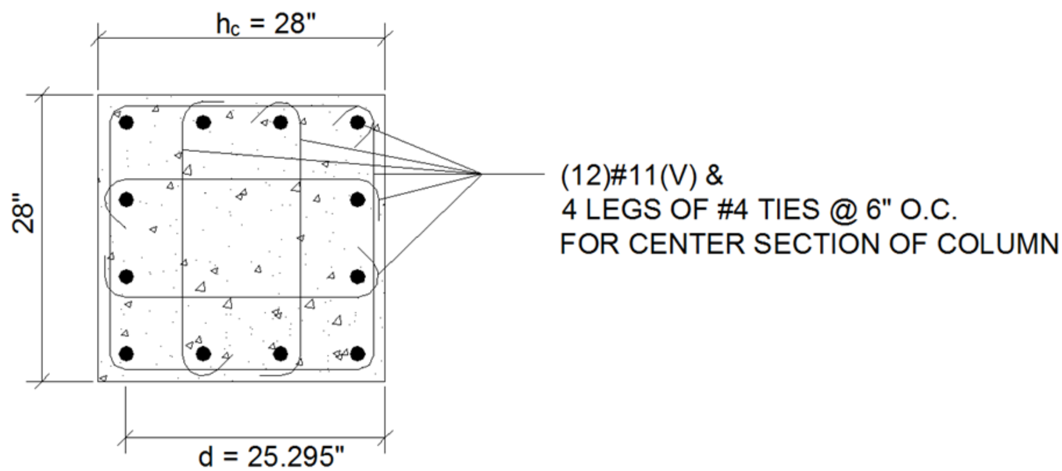

Figure A-48: Exterior column, cross-section at center section of column at the 2nd floor level based on tsunami design requirements.

A.11.1.4 Exterior Column Shear Design

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 251.5 k:

The shear capacities of the 28"x28" columns with 4 leg #5 Stirrups at 4" o.c. in the end section and 4 leg #4 Stirrups at 6" o.c. in the center section are given by:

$ØV_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{6{,}000}\left(1 + \frac{251{,}500}{2{,}000 \times 28 \times 28}\right)28 \times 25.295/1{,}000$ = 127 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4 \times 0.31) \times 60{,}000 \times 25.295}{4 \times 1{,}000}$ = 470 kips or

$V_S = \frac{A_v f_y d}{s}$ = 470 kips $\ngtr 8\sqrt{f_C'}$ b d $= 8 \times \sqrt{6{,}000} \times 28 \times 25.295 = 439$ kips $\therefore$ use 439 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4 \times 0.2) \times 60{,}000 \times 25.295}{6 \times 1{,}000}$ = 202 kips

$V_S = \frac{A_v f_y d}{s}$ = 202 kips $\ngtr 8\sqrt{f_C'}$ b d $= 8 \times \sqrt{6{,}000} \times 28 \times 25.295 = 439$ kips $\therefore$ use 202 kips

therefore in the end sections, $ØV_n$ = 0.75 (127 + 439) = 425 k

therefore in the center sections, $ØV_n$ = 0.75 (127 + 202) = 247 k

At d: V_u = 392 k < ϕV_n = 425 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 247 k ≤ ϕV_n = 247 k, therefore the column is adequate for shear at the center.

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = 213.25 k:

The shear capacities of the 28"x28" columns with 4 leg #5 Stirrups at 4" o.c. in the end section and 4 leg #4 Stirrups at 6" o.c. in the center section are given by:

$ØV_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{296{,}250}{2{,}000 \times 28 \times 28}\right)28 \times 25.295/1{,}000$ = 107 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4 \times 0.31) \times 60{,}000 \times 25.295}{4 \times 1{,}000}$ = 470 kips

$V_S = \frac{A_v f_y d}{s}$ = 470 kips $\ngtr 8\sqrt{f_C'}$ b d $= 8 \times \sqrt{4{,}000} \times 28 \times 25.295 = 358$ kips $\therefore$ use 358 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4 \times 0.2) \times 60{,}000 \times 25.295}{6 \times 1{,}000}$ = 202 kips

$V_S = \frac{A_v f_y d}{s}$ = 202 kips $\ngtr 8\sqrt{f_C'}$ b d $= 8 \times \sqrt{6{,}000} \times 28 \times 25.295 = 458$ kips $\therefore$ use 202 kips

therefore in the end sections, $ØV_n$ = 0.75 (107 + 358) = 349 k

therefore in the center sections, $ØV_n$ = 0.75 (107 + 202) = 231 k

At d: V_u = 333 k < ϕV_n = 349 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 188 k < ϕV_n = 231 k, therefore the column is adequate for shear at the center.

Critical Shears in Columns at 3rd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 233k:

The shear capacities of the 28”x28” columns with 4 leg #5 Stirrups at 4” o.c. in the end section and 4 leg #4 Stirrups at 6” o.c. in the center section are given by:

$$\text{Ø}V_n = \text{Ø}(V_C + V_S)$$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{233{,}000}{2{,}000\times28\times28}\right)28\times25.436/1{,}000$ = 103 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times25.436}{4\times1{,}000}$ = 229 kips

$$V_S = \frac{A_v f_y d}{s} = 229 \text{ kips} \ngtr 8\sqrt{f_C'}\,\mathrm{b\ d} = 8\times\sqrt{4{,}000}\times28\times25.436 = 360 \text{ kips} \therefore \text{use } 229 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times25.436}{6\times1{,}000}$ = 84 kips

$$V_S = \frac{A_v f_y d}{s} = 84 \text{ kips} \ngtr 8\sqrt{f_C'}\,\mathrm{b\ d} = 8\times\sqrt{4{,}000}\times28\times25.436 = 360 \text{ kips} \therefore \text{use } 84 \text{ kips}$$

therefore in the end sections, $\text{Ø}V_n$ = 0.75 (103 + 229) = 349 k

therefore in the center sections, $\text{Ø}V_n$ = 0.75 (103 + 84) = 141 k

At d: V_u = 107 k < ϕV_n = 250 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 80 k < ϕV_n = 141 k, therefore the column is adequate for shear at the center.

By inspection the remaining columns are adequate to resist the tsunami shear force.

A.11.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure A-15**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 24 in square column cross section shown in **Figure A-49** and **Figure A-50** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force for the end section of the column occurs at a distance "*d*" from the ends of the column, where *d* = 24 - 1.5 - 0.5 - 0.5 = 21.5 in. The critical shear force for the center section of the column occurs at "d + h_c" from the end of the column, where d + h_c = 21.5 + 24 = 45.5 in.

Floor 1 – 6

End Section (A)

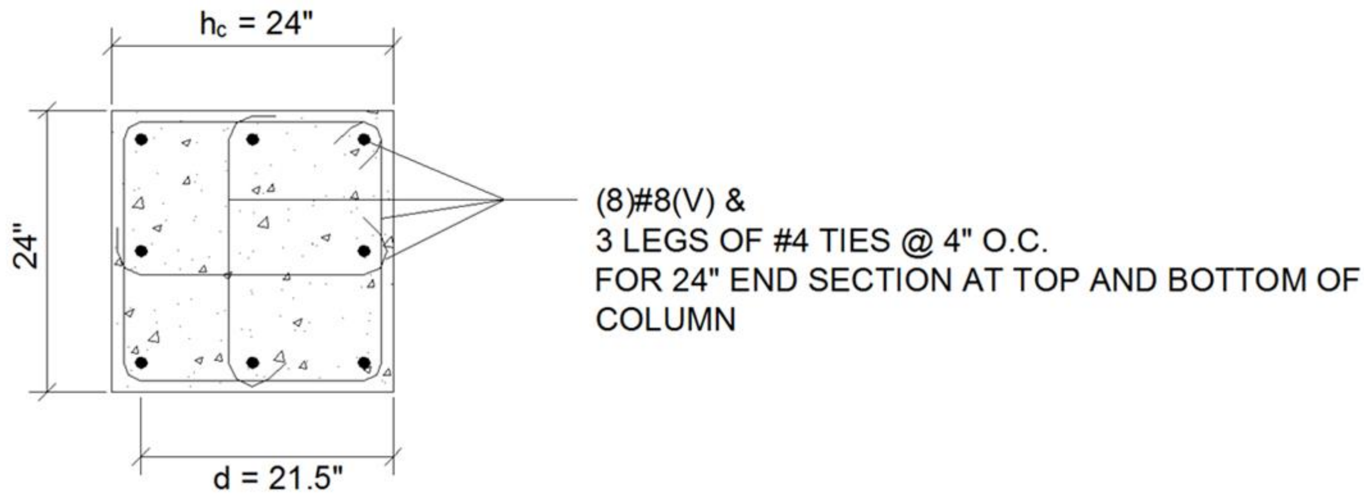

Figure A-49: Interior column, end section cross-section for column at all floor levels based on SDC D design.

Center Section (B)

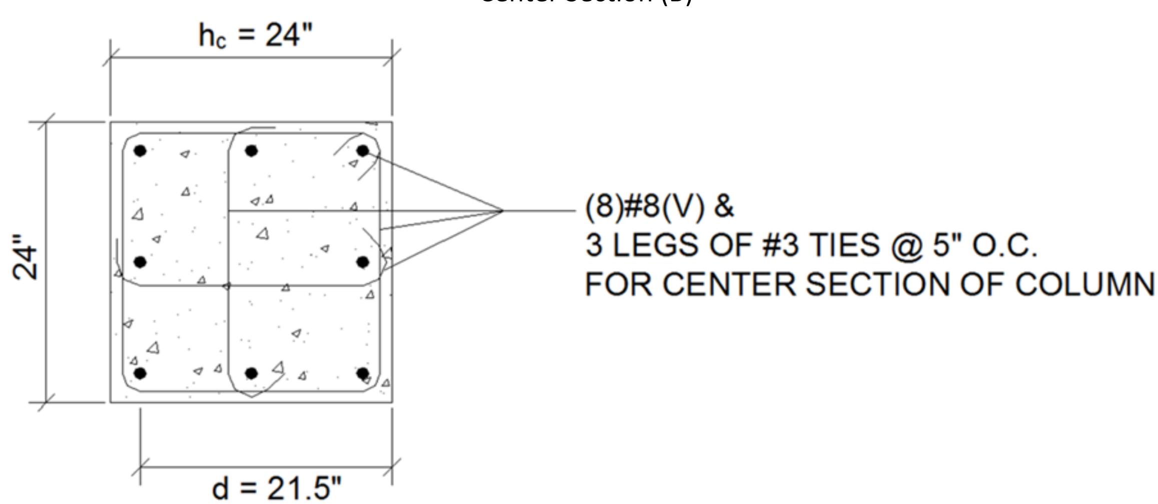

Figure A-50: Interior column, center section cross-section for of column at all floor levels based on SDC D design.

A.11.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 29 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$P_D = [120(28)(29)(6) + 2^2(150)(74)]/1000 = 629\ k.$

Floor Live load reduction factor $= 0.25+15/[4(29)(28)(5)]^{0.5} = 0.367$, *therefore using 0.4 gives:*

$P_L = 0.4[95(5)+65(24)](28)(5)]/1000 = 114\ k.$

Roof Live Load reduction factor $= R_1R_2 = 0.6(1.0) = 0.6$ *for* $A_t > 600$ *sf, therefore the roof live load is:*

$P_{Lr} = 0.6(20)(28)(29) = 9.7\ k$

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

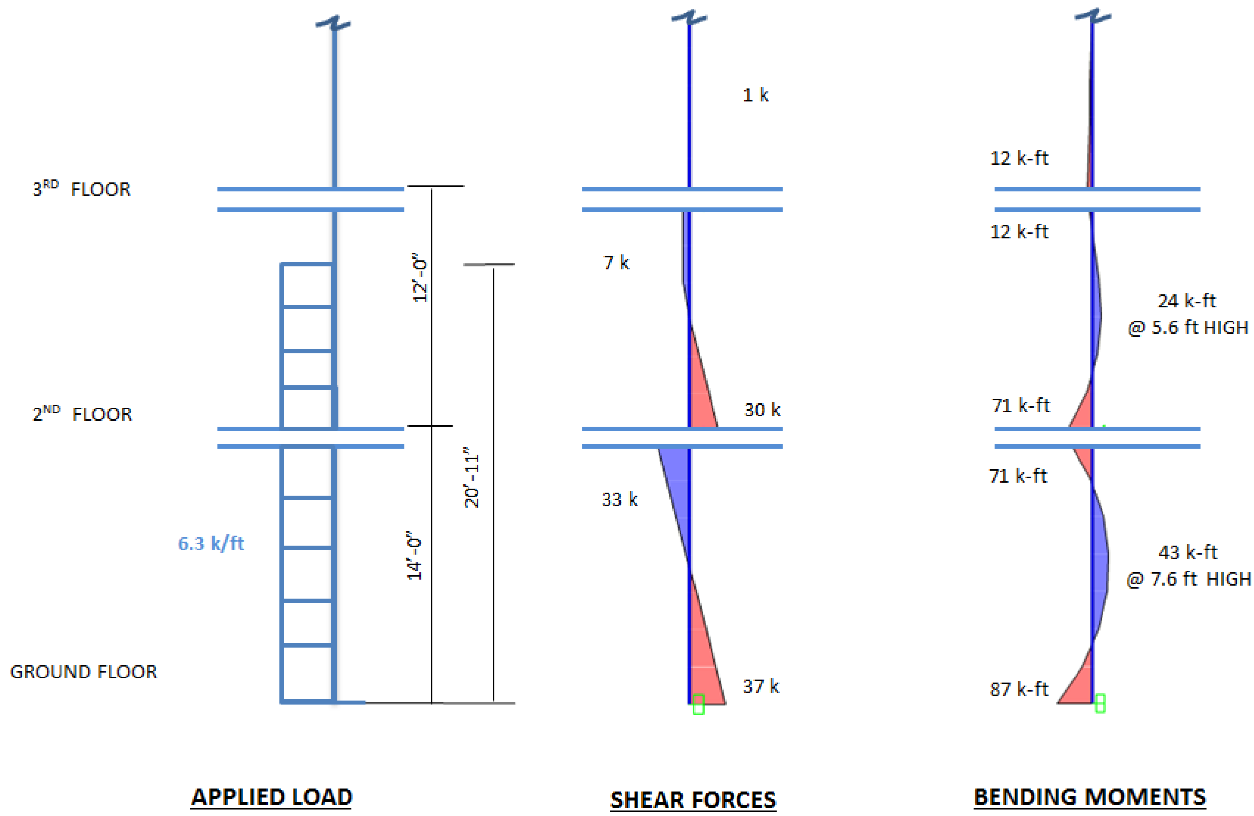

Figure A-51: Hydrodynamic loading on interior column of Seaside office building due to Load Case 2

Table A-5 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table A-5: Results from loading conditions of Seaside office building interior column

Moment	Axial Load	Shear @ d	Shear @ d + h_c	Load Combination
k-ft	Kips	Kips	Kips	
Floor 1				
87	811.8	28	17	1.2D+Ftsu+0.5L (Hydro)
87	566.1	28	17	0.9D+Ftsu (Hydro)
Floor 2				
71	676.5	20	10	1.2D+Ftsu+0.5L (Hydro)
71	471.75	20	10	0.9D+Ftsu (Hydro)
Floor 3				
12	541.2	1	1	1.2D+Ftsu+0.5L (Hydro)
12	377.4	1	1	0.9D+Ftsu (Hydro)
Floor 4				
3	405.9	0	0	1.2D+Ftsu+0.5L (Hydro)
3	283.05	0	0	0.9D+Ftsu (Hydro)
Floor 5				
1	270.6	0	0	1.2D+Ftsu+0.5L (Hydro)
1	188.7	0	0	0.9D+Ftsu (Hydro)
Floor 6				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.35	0	0	0.9D+Ftsu (Hydro)

A.11.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure A-15** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure A-52 shows the interaction diagram for a typical interior column with the tsunami load combinations.

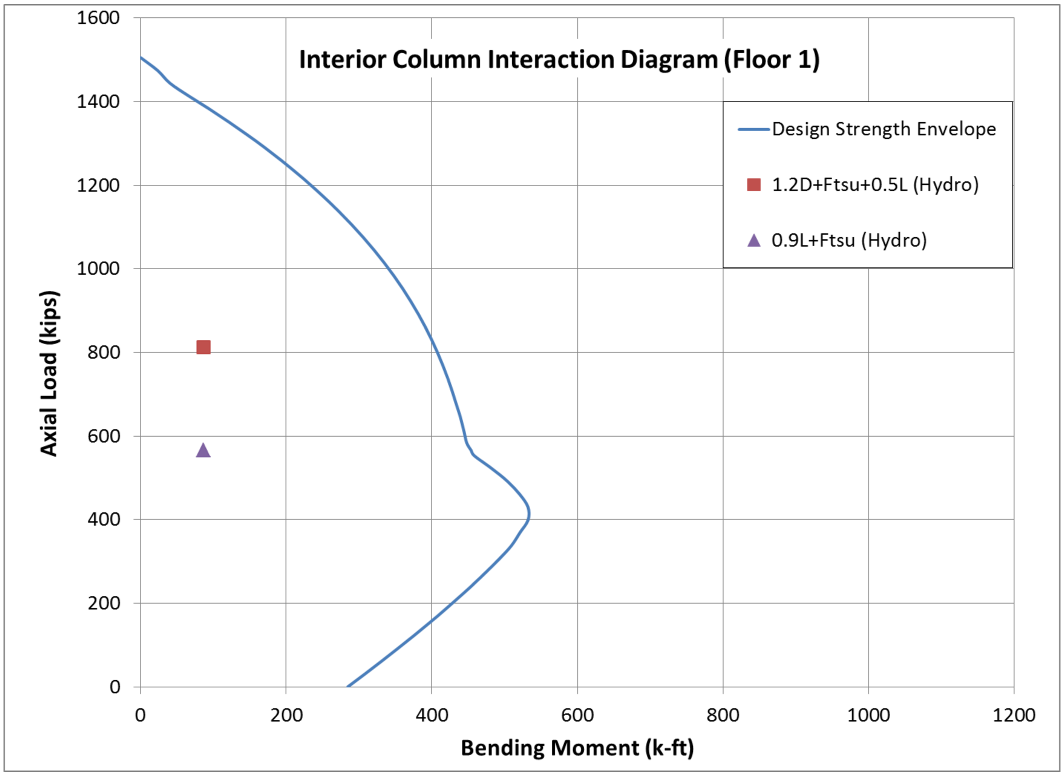

Figure A-52: Interaction diagram for typical ground floor office interior column showing tsunami load combinations

The existing interior column is therefore adequate at the first floor level, and by inspection the remaining columns are also adequate to resist the tsunami bending moments.

A.11.2.3 Interior Column Shear Design

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of ($1.2D + F_{TSU} + 0.5L$) per **Eqn. 6.8.3.3.-1a**, P_u = 811.8 kips.

The shear capacities of the existing 24"x24" column with 3 leg #4 Stirrups @ 4" o.c. in the end sections and 3 leg #3 Stirrups @ 5" o.c. in the center section are given by:

$$\emptyset V_n = \emptyset(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{811{,}800}{2{,}000\times24\times24}\right)24\times21.5/1{,}000 = 111$ kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times21.5}{4\times1{,}000} = 194$ kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times21.5}{5\times1{,}000} = 85$ kips

Therefore in the end sections, $\emptyset V_n = 0.75\,(111 + 194) = 229$ k and in the center sections, $\emptyset V_n = 0.75\,(111 + 85) = 147$ k

At d: $V_u = 43$ k < $\phi V_n = 229$ k, therefore the column is adequate for shear at the edge

At $d + h_c$: $V_u = 26$ k < $\phi V_n = 147$ k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear force.

A.12 Tsunami Design for Residential Building

A.12.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

In lieu of performing detailed hydrostatic and hydrodynamic analysis, ***Eqn. 6.10.1-1*** *provides a simplified but conservative estimate of the maximum lateral load on the building.*

$$f_{uw} = 2.5 I_{tsu} \gamma_s h_{max}^2 = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 31.4^2 = 173.53\ kip/ft$$

Assuming $C_{cx} = 0.7$ *controls per* ***Section 6.8.7***

Therefore $F = 0.7 \times 254 \times 173.53 = 30{,}854\ kips$

This lateral load can be compared with 0.75 $\Omega_o E_h$ *= 0.75 x 2.5 x 2,435 = 4,566 kips < 30,854 kips. Therefore the LFRS is not adequate to satisfy this requirement and the detailed analysis for LC2 and LC3 shown below is recommended. The components can also be designed on the basis of this conservative uniform distributed force with the appropriate width b dimensions (but that is not illustrated here).*

A.12.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

A.12.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} ≤ h_{max} = 31.4 ft

≤ 14 ft (ground floor story height)

≤ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

Load Case 1 also requires application of the associated hydrodynamic drag on the entire building. However this will not control since buoyancy need not be considered.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4575 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254' overall width of building

h = 8'

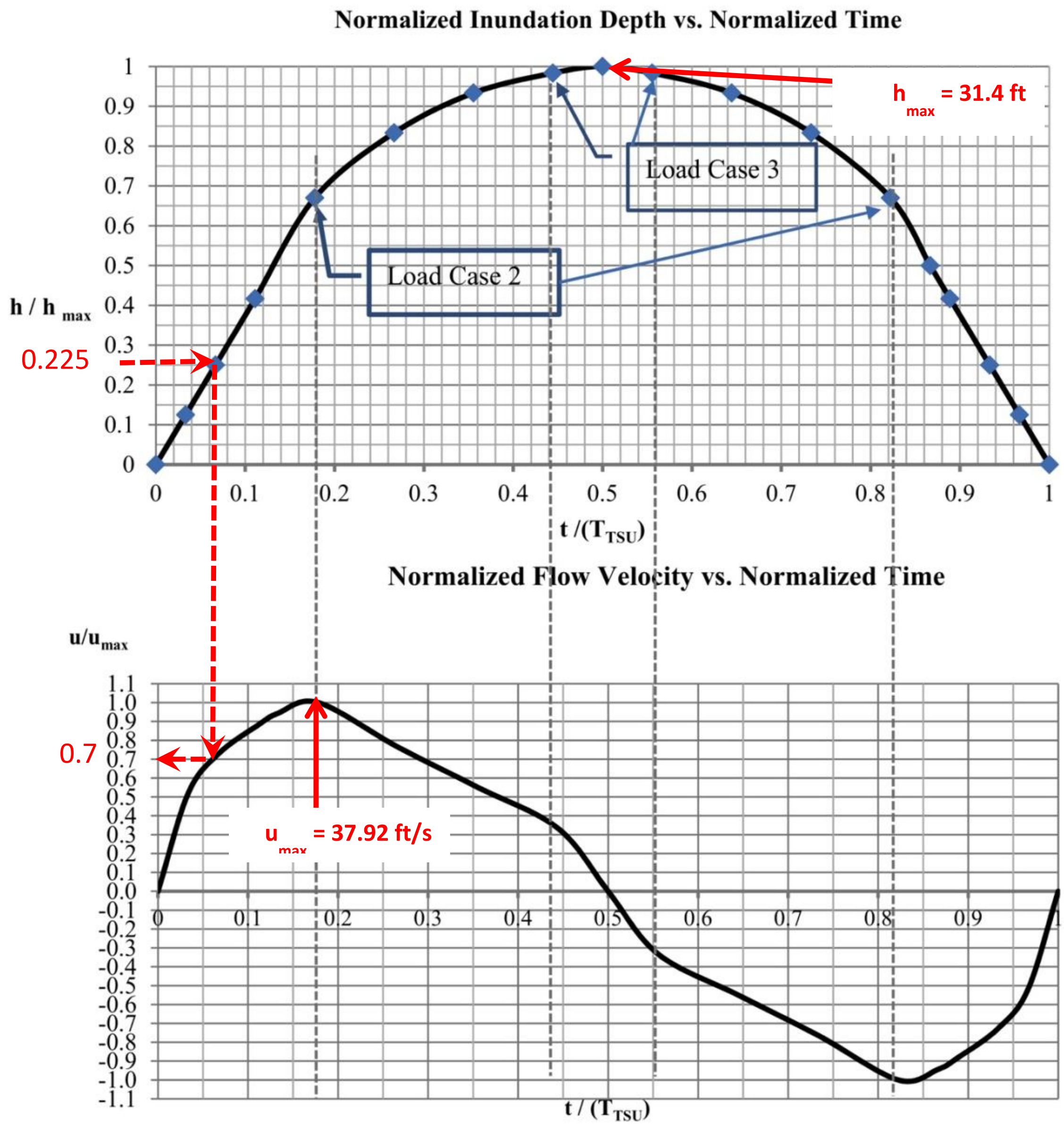

Figure A-53: Determining "u" for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/31.4 = 0.255. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.068. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.7. Therefore the flow velocity is u = 0.7 x 37.92 = 26.54 fps.

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 26.54^2)/1000 = 2{,}295\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail to relieve lateral load on the structural frame. Note that a portion of this load will go to the ground floor slab, which reduces the load that has to be resisted by the lateral force resisting system. The entire lateral load must be resisted by the deep foundations assuming maximum scour has already occurred.

A.12.2.2 Load Case 2: Maximum Flow Velocity

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is $2/3h_{max}$ = 2/3 x 31.4 = 20.93 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.252 (**Table 6.10-1** based on B/h_{sx} = 254/20.93 = 12.136)

Since the inundation depth of 20.93 feet exceeds the bottom of the third floor slab (12'+9' – 8"/12) = 20.33', the inundated area of the beams must be included in the closure coefficient, which is determined as follows:

h_{sx} = 20.93 ft.

A_{col} = 32 x 1.67' x (20.93'-1.9 x 0.67') = 1049 ft^2

A_{wall} = 2 x 28' x (20.93'-1.9 x 0.67') + 2 x 10' x (20.93'-1.9 x 0.67') = 1495 ft^2

A_{Beam} = A_{Slab} = 1.9 x 254' x 0.67' = 322 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5\times A_{beam}}{Bh_{sx}} = \frac{\sum((1049+1495)+1.5\times 322}{254\times 20.93} = 0.569 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

$u = u_{max}$ = 37.92 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.251 \times 0.7 \times 254(20.93 \times 37.92^2)/1000 =$ 7,369 $kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 20.93 feet above grade. The lateral force resisting system for the structure must be evaluated for this load. During drawdown the same pressure needs to be applied to the inland elevation and the lateral force resisting system evaluated for this load.

A.12.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 31.4 ft. and the flow velocity is $1/3u_{max}$ = 1/3 x 37.92 = 12.64 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/31.4 = 8.09)

Since the inundation depth of 31.4 ft exceeds the fourth floor slab elevation of 30 ft, the closure coefficient is given by:

h_{sx} = 31.4 ft.

A_{col} = 32 x 1.67' x (31.4'-0.67'-0.67') = 1568 ft^2

A_{wall} = 2 x 28' x (31.4'-0.67'-0.67'-0.67') + 2 x 10' x (31.4'-0.67'-0.67'-0.67') = 2235 ft^2

A_{Beam} = A_{Slab} = 2 x 254' x 0.67' = 508 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum(1568+2235)+1.5\times 508}{254' \times 31.4'} = 0.572 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 12.64 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(12.64 \times 31.4^2)/1000 = 1{,}226\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 31.4 feet above grade. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth.

A.12.2.4 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category D, **Section 6.8.3.4** permits the use of $0.75\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear. From the seismic design of this structure, E_h = 2,435 kips. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 2.5 \times 2{,}435 = 4{,}566\ kips$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 7,368 kips applied over a height of 20.93 ft. A portion of this load will be resisted by the grade beam/foundation system, reducing the overall load by 2,112 kips. (**Figure A-54**)

$$0.75\Omega_o E_h = 4{,}566 kips < 5{,}256\ kips$$

Therefor the lateral force resisting system is not adequate and the ETABS model needs to be reevaluated.

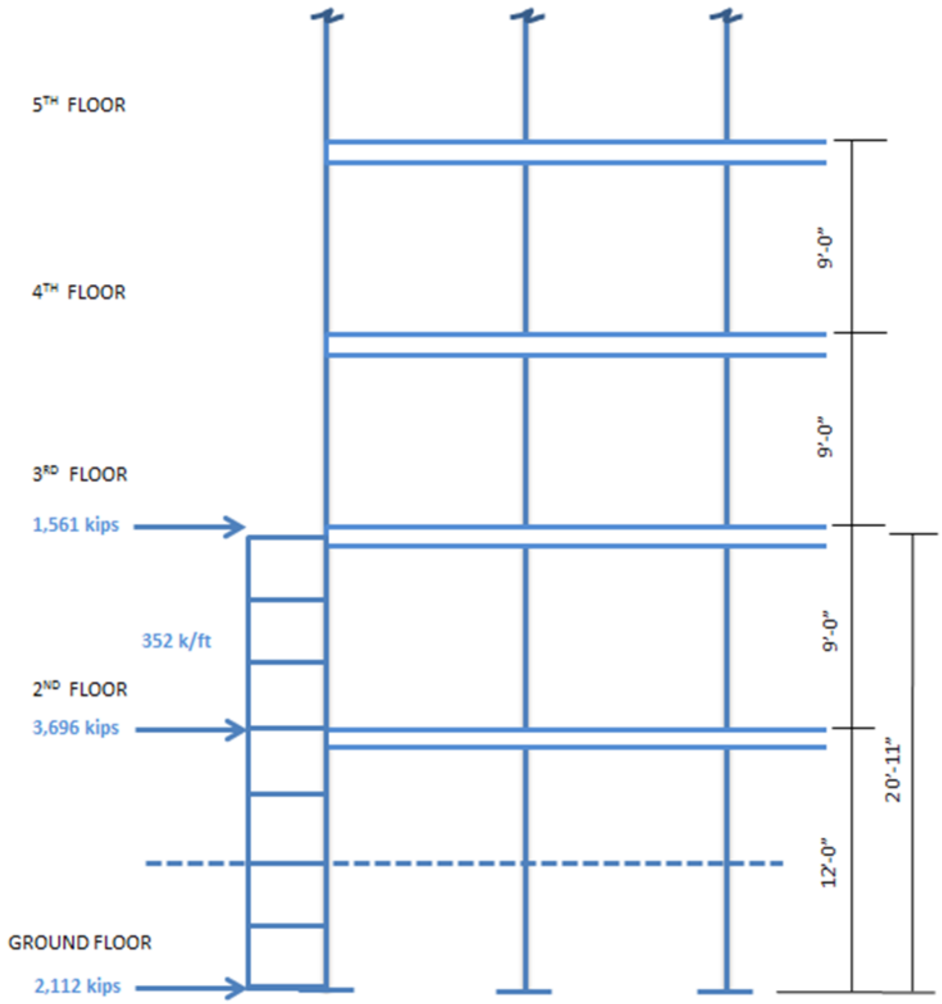

Figure A-54: LC2 Tsunami loads on overall Seaside Residential building

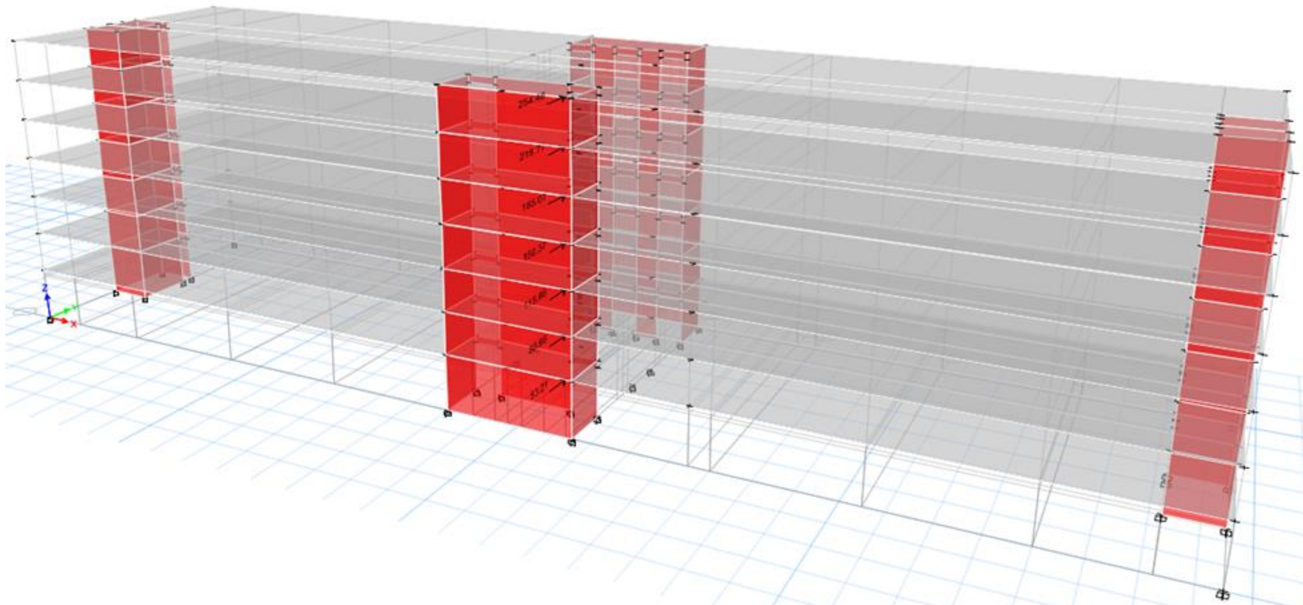

Figure A-55: ETABS computer model of residential building subjected to elevated seismic loads to meet the tsunami demand at the Seaside location

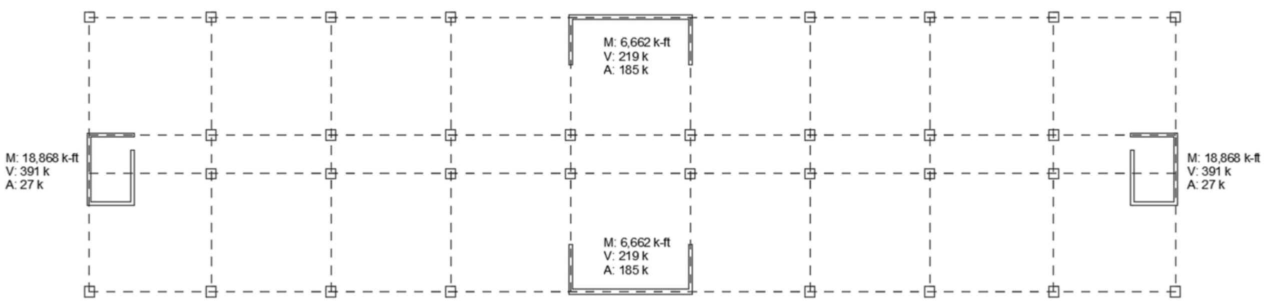

Figure A-56: ETABS output of axial load, shear force and bending moment at the base of each structural wall ground floor

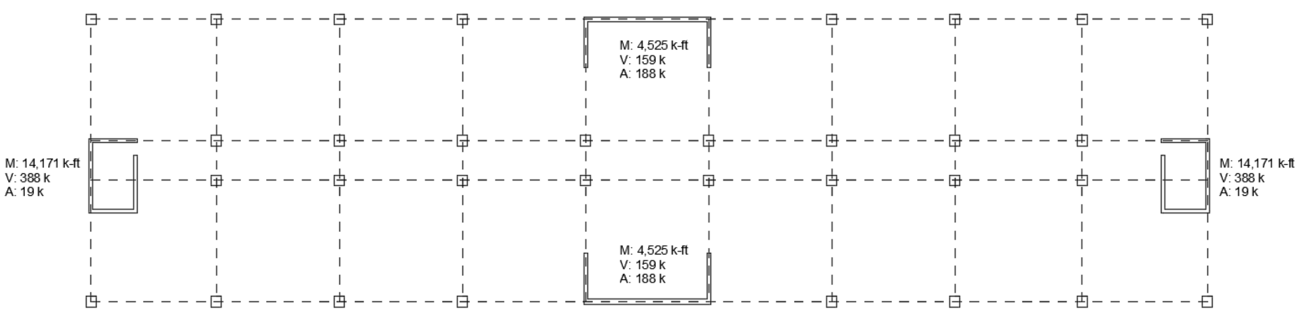

Figure A-57: ETABS output of axial load, shear force and bending moment at the base of each structural wall 2nd floor

A.13 Component Design

A.13.1 Drag Force on Components - Section 6.10.2.2

A.13.1.1 Exterior Columns

Exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** requires that C_d = 2.0 and the width dimension, *b*, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Therefore *b* = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 20.93 ft and u_{max} = 37.92 fps.

The hydrodynamic drag is computed using **Eqn 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(20.93 \times 37.92^2)/1000 = 1{,}298\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 1,298/20.93 = 62 kips/ft over the lower 20.93 feet of the column. The column must be designed for this load combined with gravity loads per **Section 6.8.3.3**.

A.13.1.2 Interior Columns

Interior columns are 20" (1.67 ft) square R.C. columns. The controlling load case will be LC2, when the inundation depth is is h_e = 20.93 ft and u_{max} = 37.92 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**) and b = 1.67 ft since no debris accumulation is considered for interior column.

Therefore $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 1.67(20.93 \times 37.92^2)/1000 = 110.4\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 110.4/20.93 = 5.27 kips/ft over the lower 20.93 feet of the column. This load must be combined with gravity loads per **Section 6.8.3.3** and the column capacity verified.

A.13.2 Tsunami Loads on Structural Walls, F_w – Section 6.10.2.3

Since tsunami bores are anticipated at this location, the lateral load on the structural walls is given by **Eqn. 6.10-5a** or **Eqn. 6.10-5b**, depending on the flow depth relative to the wall width:

Eqn. 6.10-5a $F_W = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$

Eqn. 6.10-5b $F_W = \frac{3}{4}\rho_s I_{tsu} C_d b(h_e u^2)$ when $\frac{b_w}{h_e} \geq 3$

Where C_d = 2.0 for a wall per **Table 6.10-2**, and

Elevator Walls:

b = 28' for the elevator walls

Elevator $\frac{b_w}{h_e} = \frac{28\prime}{20.93} = 1.34 \ngeq 3 \therefore \text{Eqn. 6.10-5a}$

The controlling load case will be LC2, where h_e = 20.93 ft and u = 37.92 fps.

Therefore, for the 28' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 28(20.93 \times 37.92)/1000 = 1{,}854 kips$

These loads are applied to the walls as a uniformly distributed pressure of 1,854/(28x20.93) = 3,163 psf over the lower 20.93 ft of the walls. ← **(CONTROLS)**

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3} \rightarrow h_e \geq \frac{b_w}{3\prime} = \frac{28\prime}{3\prime}$ = 9.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 9.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure A-58**. The resulting velocity is h/h_{max} = 9.33'/31.4' = 0.297. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.08. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.78. Therefore the flow velocity is u = 0.78 x 37.92 = 29.6 fps. The bore loading is computed for h_e = 9.33 ft and u = 29.6 fps.

Therefore for the 28' wide elevator wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 28(9.33 \times 29.6^2)/1000 = 755\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 755/(28x9.33) = 2,890 psf over the lower 9.33 ft of the walls. This will not govern when compared with the pressure from hydrodynamic drag in **Eqn. 6.10-5a**.

Stairwell Walls:

b = 10' for the elevator walls

Stairwell $\frac{b_w}{h_e} = \frac{10\prime}{20.93\prime} = 0.48 \ngeq 3 \therefore \text{Eqn. 6.10-5a}$

The controlling load case will be LC2, where h_e = 20.93 ft and u = 37.92 fps.

Therefore, for the 10' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 10(20.93 \times 37.92^2)/1000 = 662\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 662/(10x20.93) = 3,163 psf over the lower 20.93 ft of the walls. ← **(CONTROLS)**

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3}$ → $h_e \geq \frac{b_w}{3'} = \frac{10'}{3'}$ = 3.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 3.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure A-58**. The resulting velocity is h/h_{max} = 3.33'/31.4' = 0.106. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.03. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.45. Therefore the flow velocity is u = 0.45 x 37.92 = 17.1 fps. The bore loading is computed for h_e = 3.33 ft and u = 17.1 fps.

Therefore for the 10' wide stairwell wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 10(3.33 \times 17.1^2)/1000 = 32.1\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 32.1/(10x3.33) = 963 psf over the lower 3.33 ft of the walls. This will not govern when compared with the pressure from hydrodynamic drag in Eqn. **6.10-5a**.

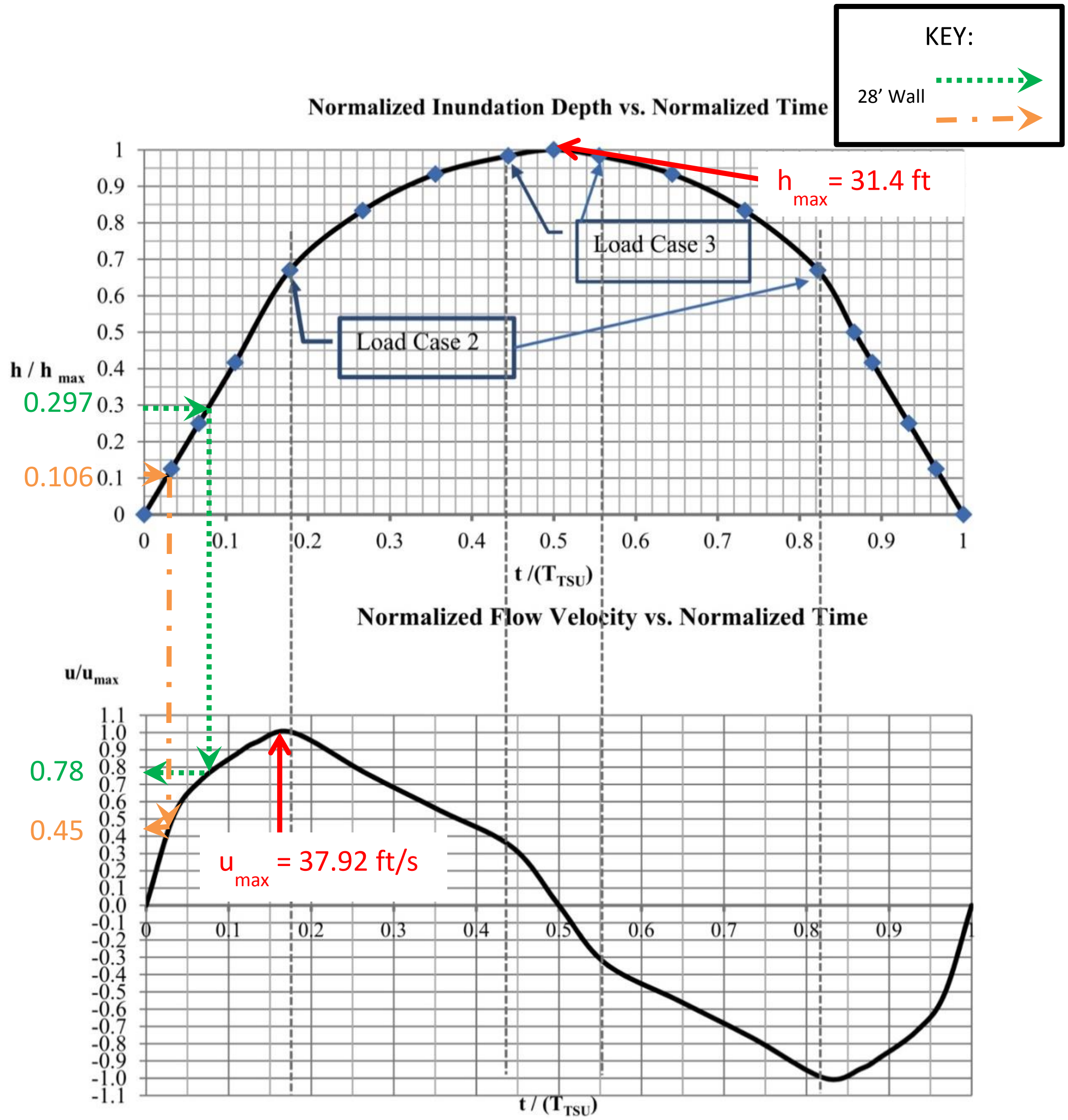

Figure A-58: Determining "u" for Eqn. 6.10-5b with ASCE 7-16 Figure 6.8-1

A.13.3 Hydrodynamic Pressures associated with Slabs – Section 6.10.3

A.13.3.1 Flow Stagnation Pressure – Section 6.10.3.1

The Mechanical/Electrical room on Gridline D between Gridlines 5 and 6 is enclosed on all sides by structural walls. Tsunami flow entering through the two door openings will result in flow stagnation pressurization of this room, given by **Eqn. 6.10-8** as:

$$P_p = \frac{1}{2}\rho_s I_{tsu} u^2$$

Assuming that the door openings are 7 ft high, the stagnation pressurization is based on the maximum flow velocity occurring at this or greater depths, ie. when the door opening is fully submerged. The flow

velocity will therefore be the maximum of 37.92 fps which occurs when the flow depth is 20.93 ft (**Figure 6.8-1**, LC2). Therefore;

$$P_p = \frac{1}{2} \times 2.2 \times 1.0 \times 37.92^2 = 1{,}582\ psf$$

The structural walls surrounding this room must be evaluated for an outward pressure of 1,582 psf, in combination with gravity loads per **Section 6.8.3.3**. The floor slab above this room must be designed for a net uplift pressure given by 0.9D + F_{TSU} = -0.9x100 + 1,582 = 1,492 psf upwards. This will require additional top reinforcement in this slab and possibly shear reinforcement around the slab perimeter. In order to reduce the amount of additional reinforcement, one could perform a non-linear analysis of the floor slab following the provisions of ASCE 41. A simpler alternative may be to design the floor slab in the mechanical room as a breakaway slab, as shown in **Figure A-59,** in order to relieve pressure. This will apply to all levels up to h_{max}

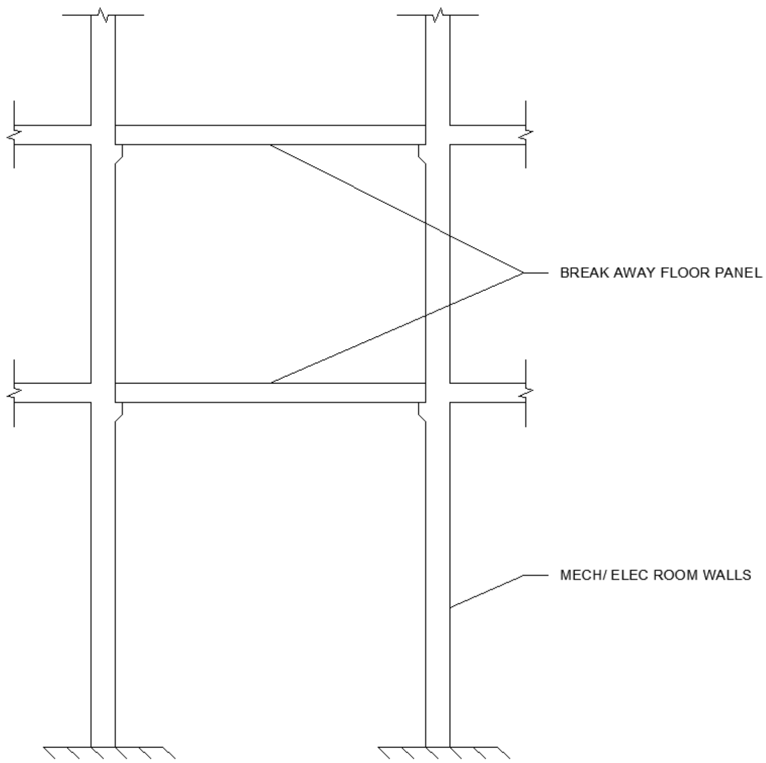

Figure A-59: Mechanical/Electrical room break-away floor panels applied to all levels up to h_{max}

A.13.3.2 Hydrodynamic Surge Uplift at Horizontal Slabs – Section 6.10.3.2

If slabs are submerged during the tsunami, they must be designed for uplift, with a specified minimum of 20 psf (**Section 6.10.3.2.1**). The uplift may increase if the ground floor is sloped, causing an upward component of flow velocity (**Section 6.10.3.2.2**). This is not the case for this building.

The resulting minimum uplift of 20 psf is much smaller than the dead weight of the slab (100 psf), therefore this uplift will not affect the slab design.

A.13.3.3 Tsunami Bore Flow Entrapped in Structural Wall-Slab Recesses – Section 6.10.3.3

If a tsunami bore is entrapped in a structural wall-slab recess, then large pressures can develop on the slab and wall (**Section 6.10.3.3.1**). Although tsunami bores are anticipated at this location, the flow can pass freely around the wall elements in this building. Therefor this condition does not apply.

A.14 Debris Impact Loads - Section 6.11

The inundation depth exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads.

A.14.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330 I_{tsu} C_o = 330 \times 1.0 \times .65 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force can be reduced to 50%, or 107.25 kips. This load must be applied to the 20" square exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with other tsunami loads and it need not be applied to interior columns.

This equivalent static impact load of 107.25 kips must also be applied to any structural walls on the perimeter of the building. This applies to the 28 ft wide elevator walls on both exterior sides of the building (GLs A and D) since impact must be considered during inflow and outflow conditions. Evaluation of the wall capacity is based on a tributary wall width of half the wall height. Since the wall unbraced height is (12' – 8"/12) = 11.33', the tributary width is 5.67 ft.

A.15 Column Design for Tsunami Loads

A.15.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 from **Figure A-16**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation, therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure A-60** and **Figure A-61** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the end of the column, where *d* = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "d + h_e" from the end of the column, where d + h_e = 17.5 + 20 = 37.5 in.

Floor 1 – 7

End Section (A)

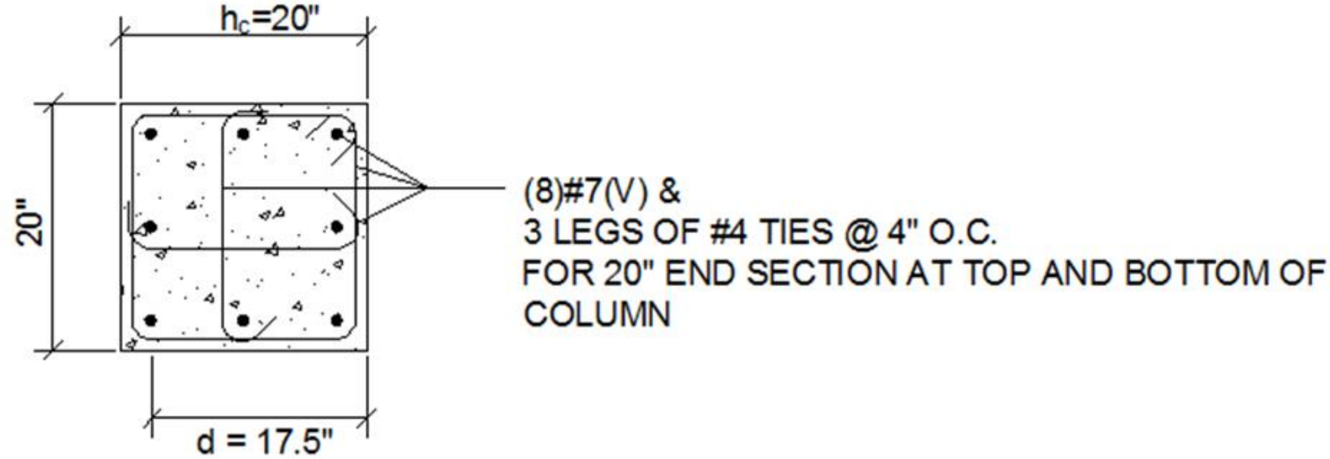

Figure A-60: Exterior column, cross-section at end of column at all floor levels based on SDC D design.

Center Section (B)

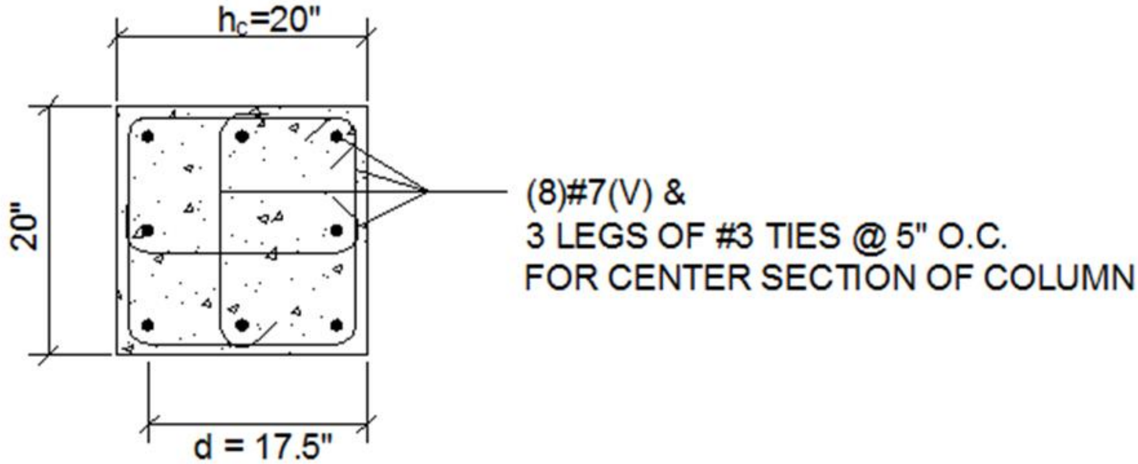

Figure A-61: Exterior column, cross-section at center of column at all floor levels based on SDC D design.

A.15.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 14.58 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

$$P_D = [128(14.58)(28)(6)+ 123(14.58)(28)+ 90(28)(6)+1.67^2(150)(66)]/1000 = 406 \text{ k}$$

Floor Live load reduction factor $= 0.25+15/[4(14.58)(28)(5)]^{0.5} = 0.402$, *therefore, column base live load is:*

$$P_L = 0.402[55(14.58)(28)(6)]/1000 = 54.2 \text{ k}$$

Roof Live Load reduction factor $= R_1R_2= [1.2-(0.001)(14.58)(28)](1.0) = 0.792$, *therefore, roof live load is:*

$P_{Lr} = 0.792(20)(14.58)(28)/1000 = 6.47kF$

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

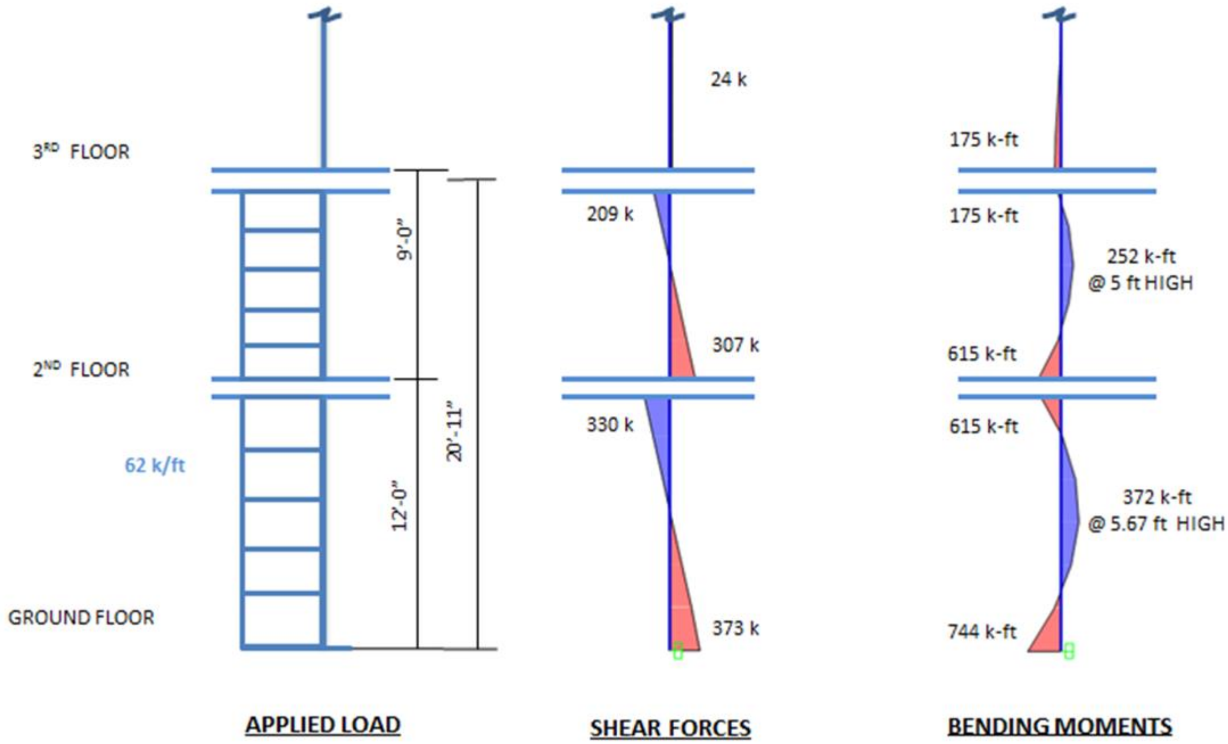

Figure A-62: Hydrodynamic loading on exterior column of Monterey residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just above and below each inundated floor slab for the maximum shear and near the mid-height of the clear column height for maximum bending moments. Samples of the resulting shear force and bending moment diagrams are provided below. Similar diagrams and similar shear and bending moments would result if the impact load was applied at the other end of each column.

Impact load at d:

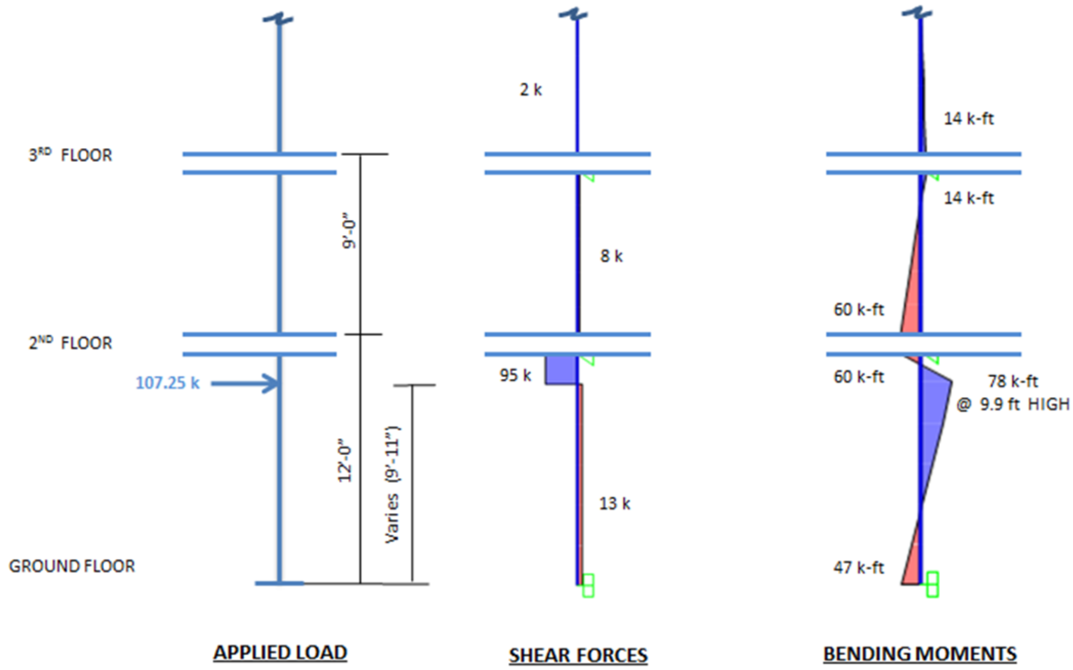

Figure A-63: Impact load applied at "d" away from the end of column on the ground floor

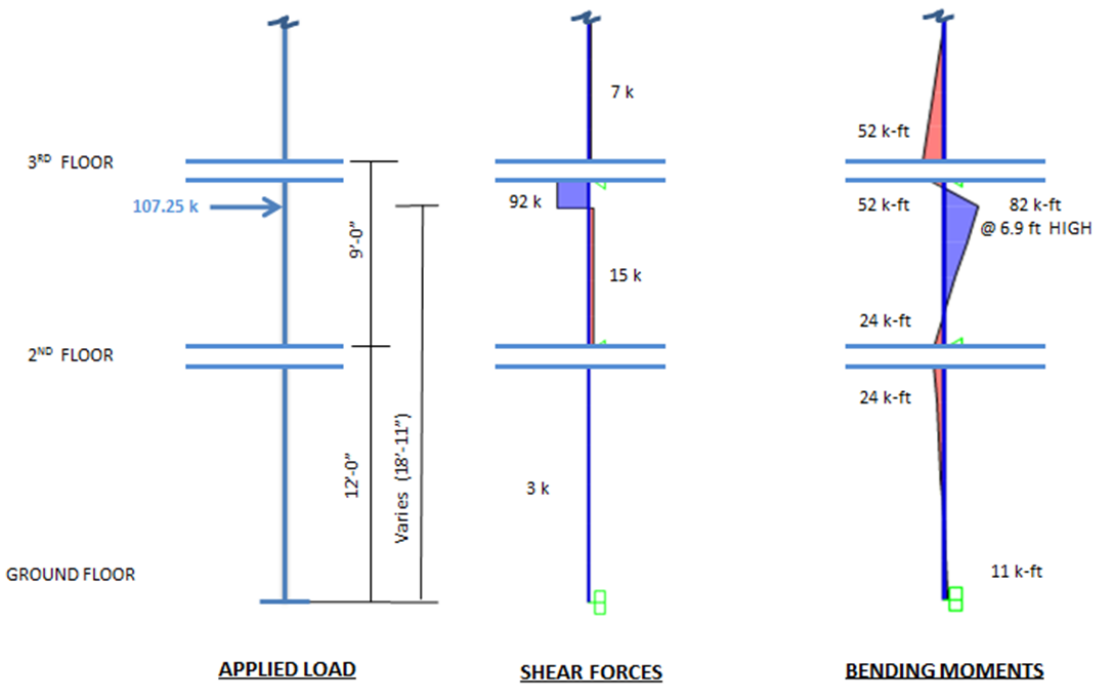

Figure A-64: Impact load applied at "d" away from the end of column on the 2nd floor

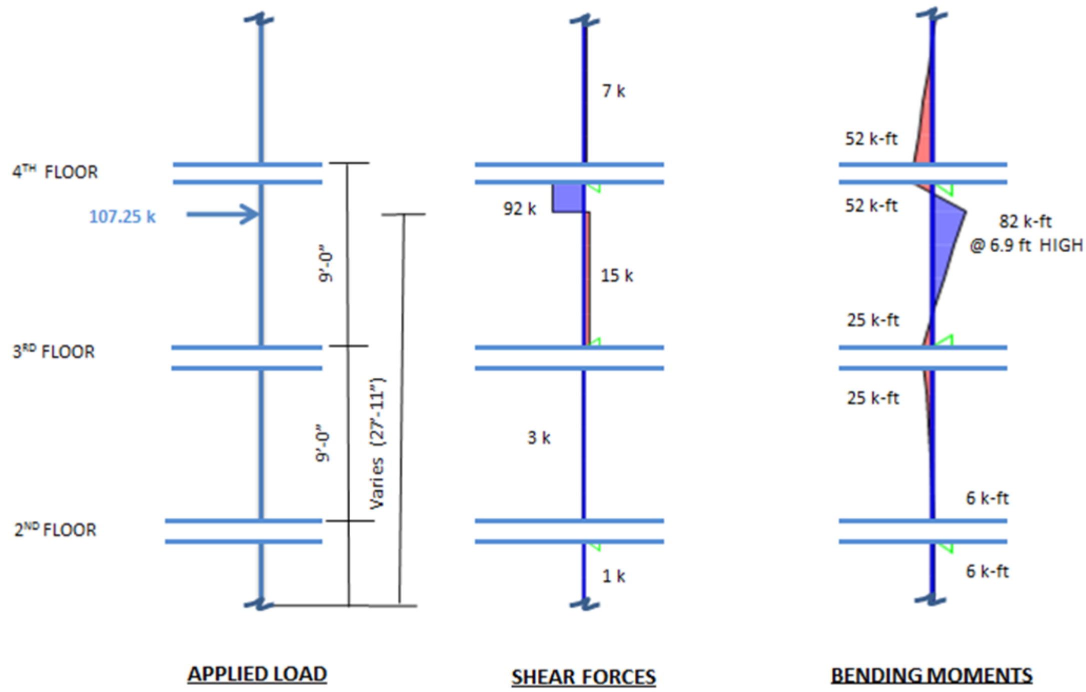

Figure A-65: Impact load applied at "d" away from the end of column on the 3rd floor

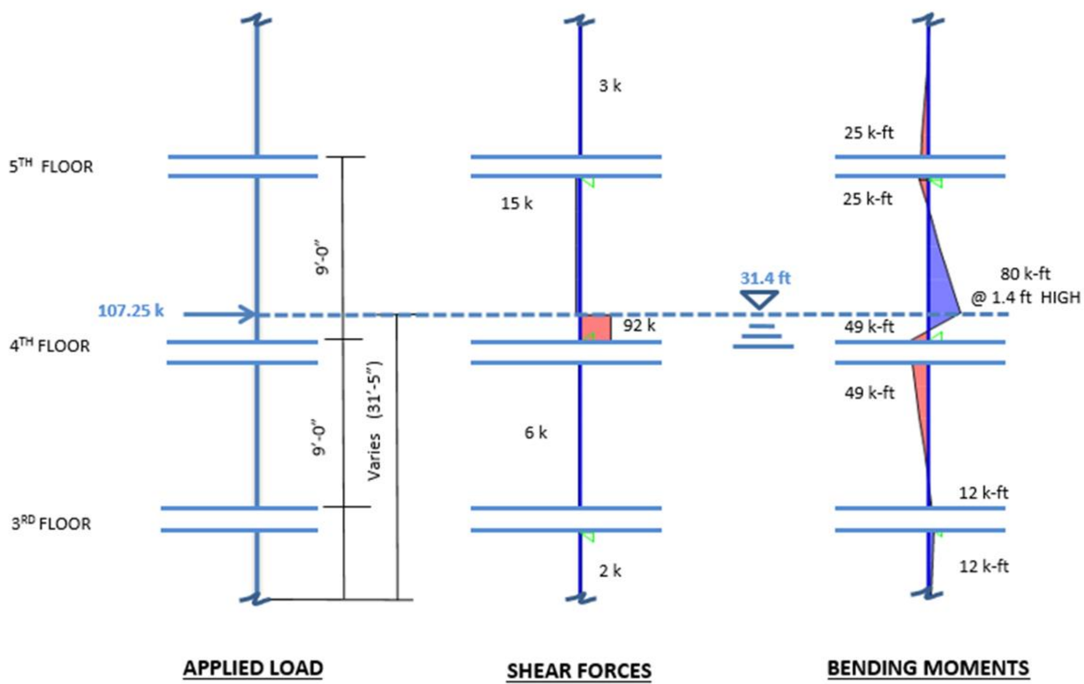

Figure A-66: Impact load applied at 1.4 ft away instead of "d" as water level is lower than "d" away from the end of column on the 4th floor

Impact load at d + h_c:

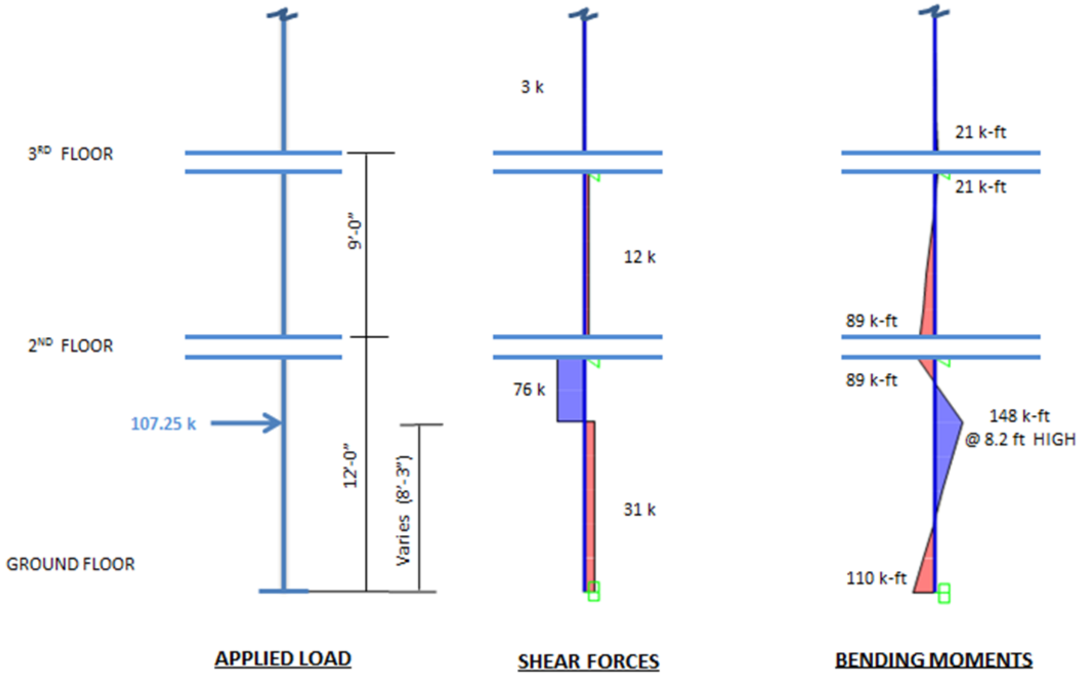

Figure A-67: Impact load applied at "d + h_c" away from the end of column on the ground floor

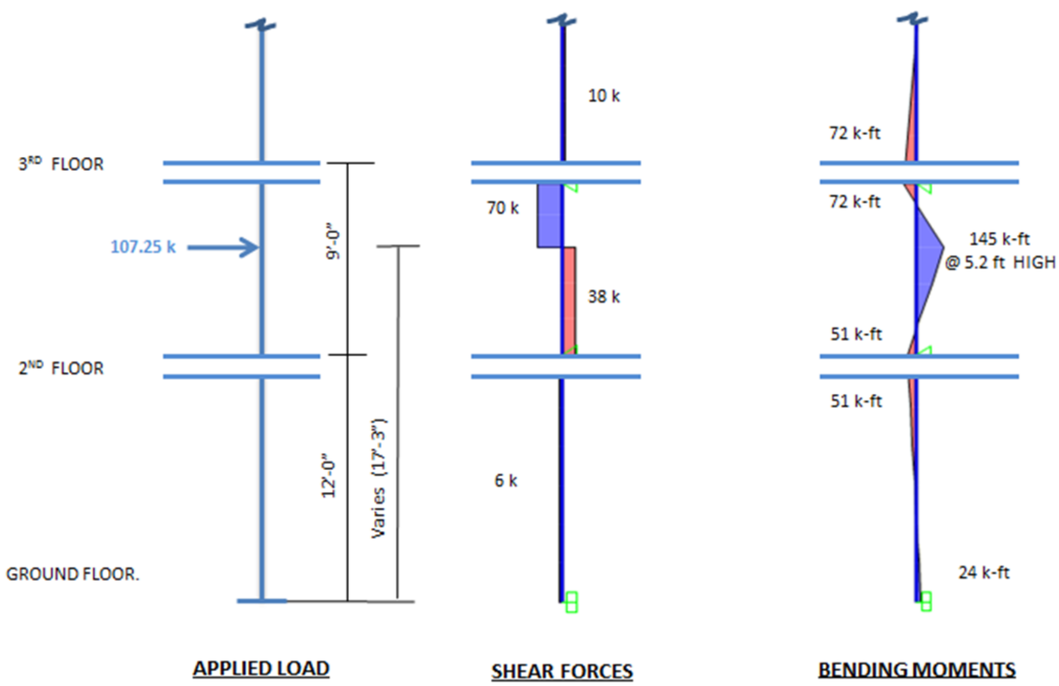

Figure A-68: Impact load applied at "d + h_c" away from the end of column on the 2nd floor

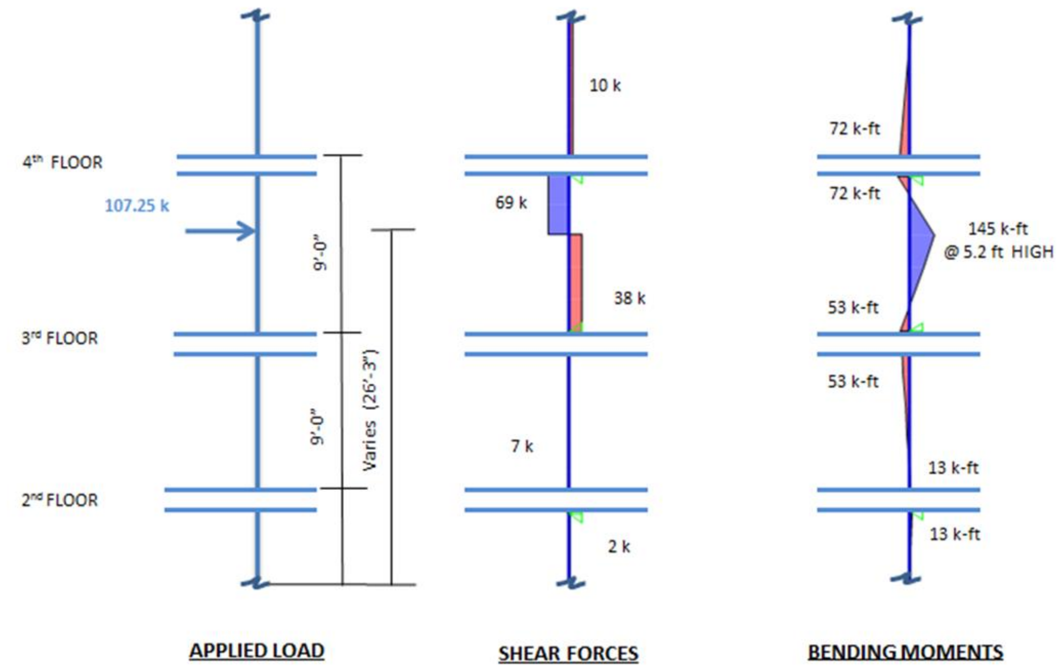

Figure A-69: Impact load applied at "d + h_c" away from the end of column on the 3rd floor

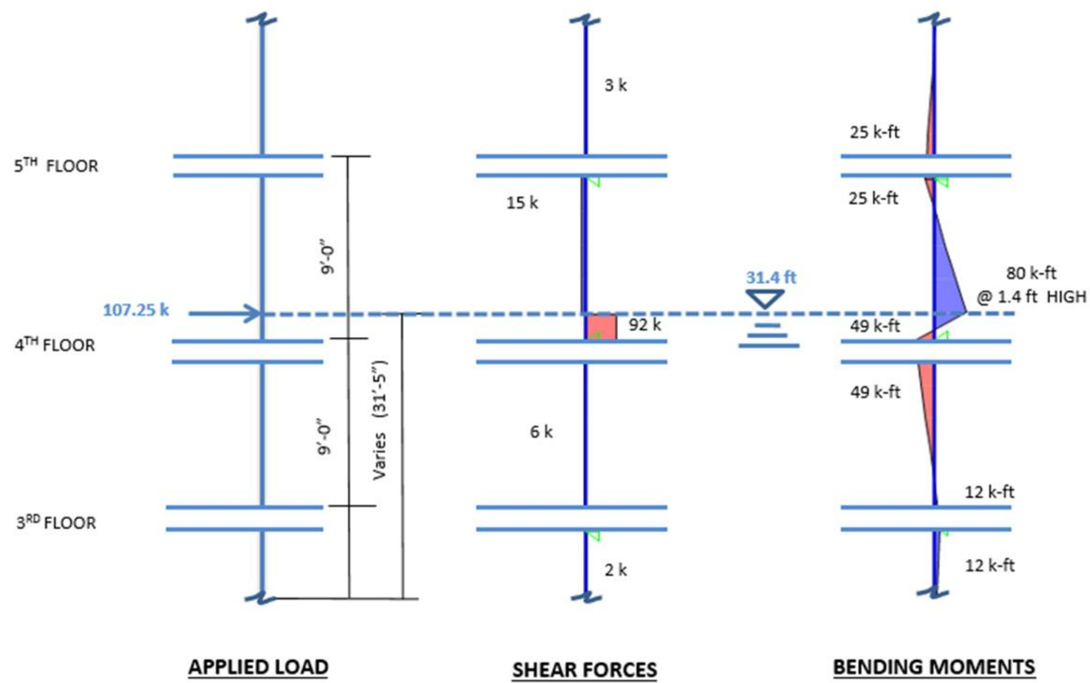

Figure A-70: Impact load applied at 1.4 ft away instead of "d" as water level is lower than "d" away from the end of column on the 4th floor

Impact load at mid-height:

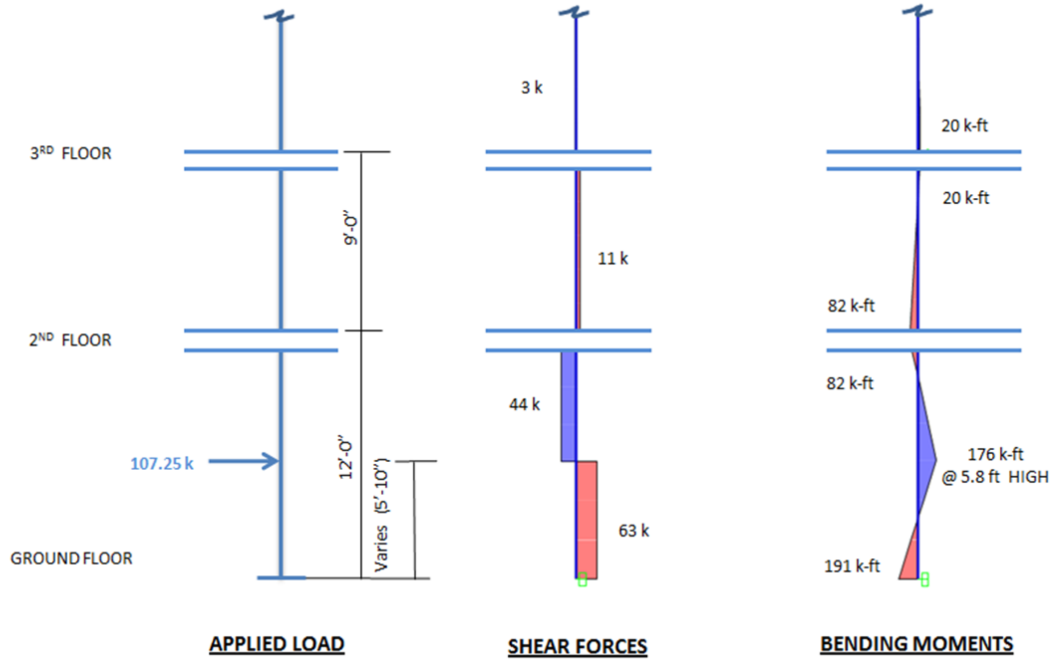

Figure A-71: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

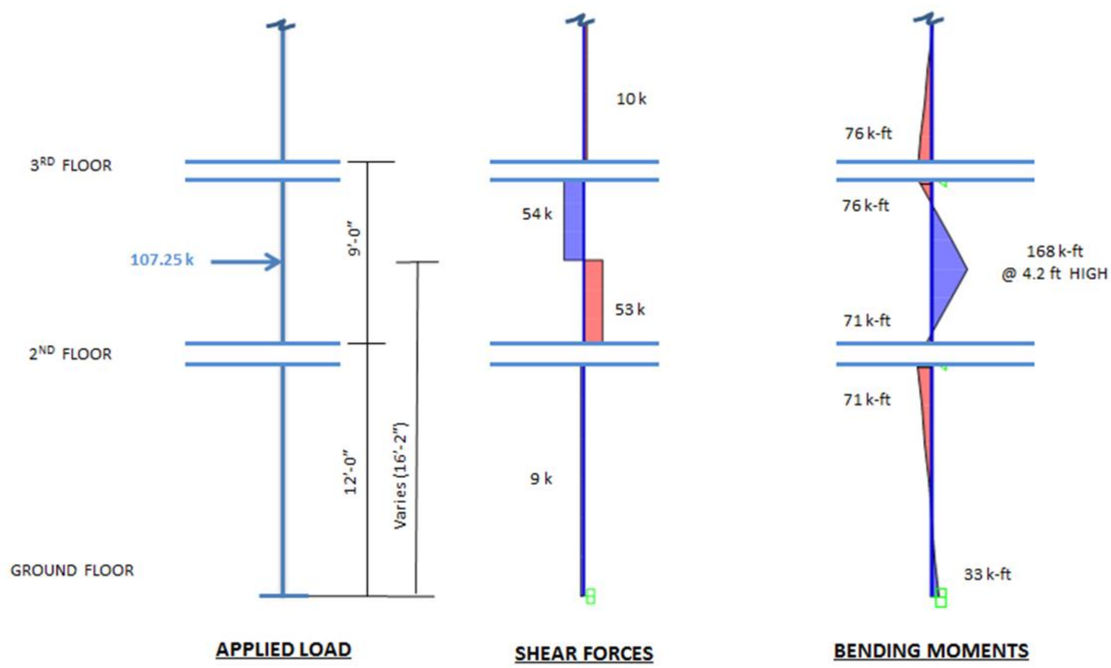

Figure A-72: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

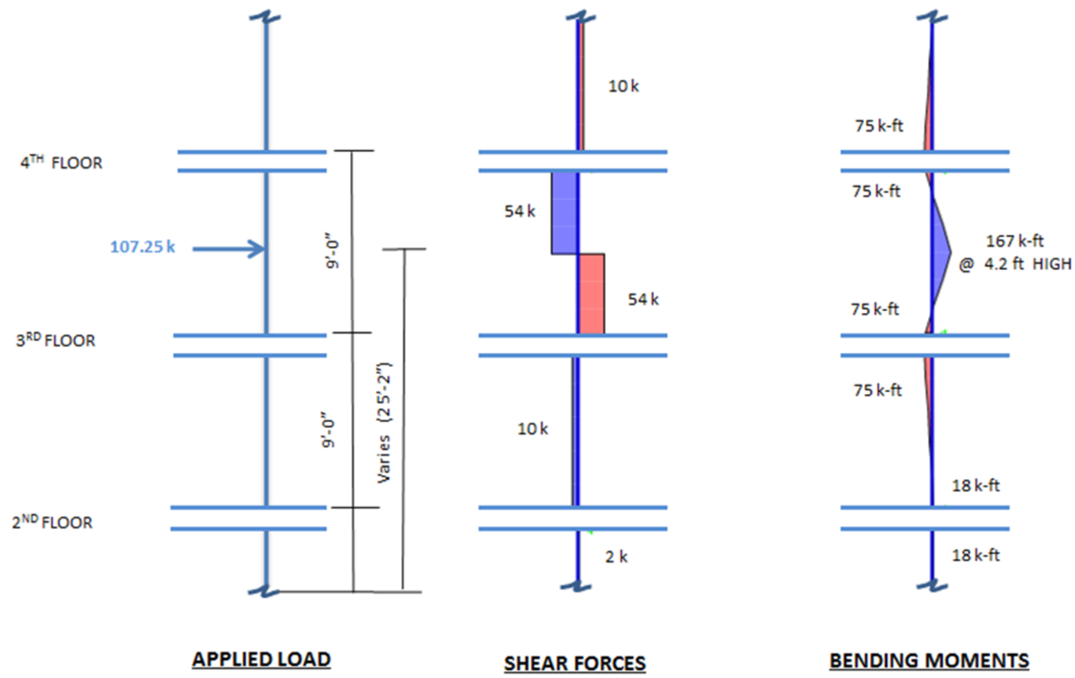

Figure A-73: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

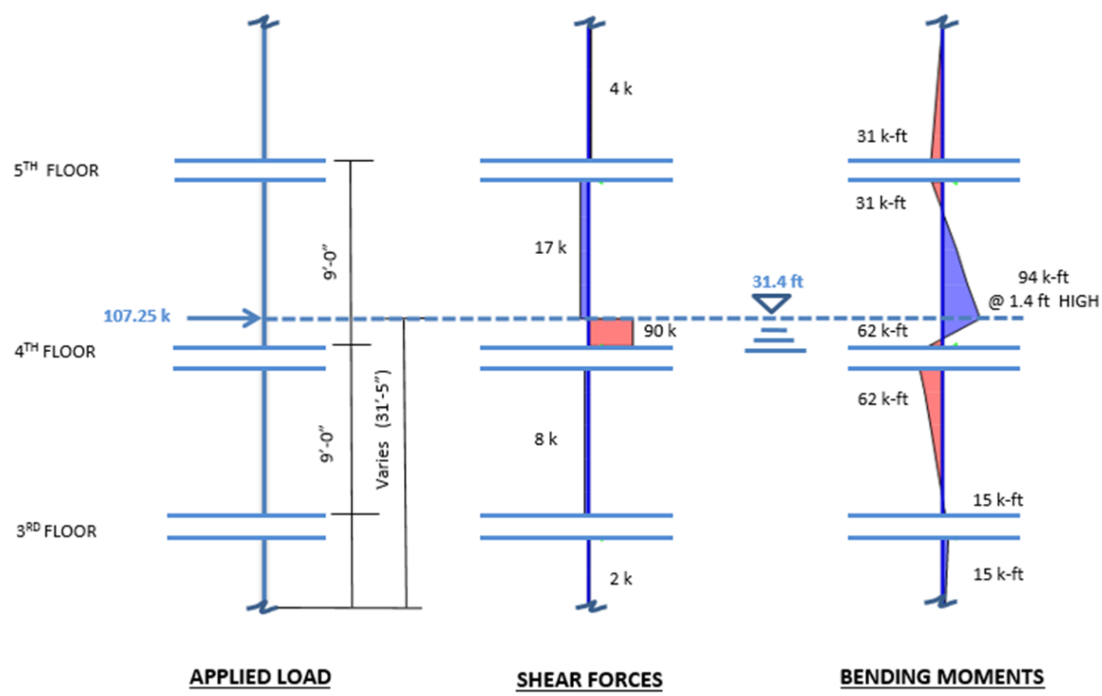

Figure A-74: Impact load applied at 1.4 ft away from the assumed lateral restraint instead of the mid-height of the assumed lateral restraint points at top and bottom of the 4th floor column

Table A-6 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro)

and long impact (Impact). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table A-6: Results from loading conditions of Seaside residential building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
744	514.3	282	179	1.2D+Ftsu+0.5L (Hydro)
744	365.4	282	179	0.9D+Ftsu (Hydro)
191	514.3	95	76	1.2D+Ftsu+0.5L (Impact)
191	365.4	95	76	0.9D+Ftsu (Impact)
Floor 2				
615	440.8	217	113	1.2D+Ftsu+0.5L (Hydro)
615	313.2	217	113	0.9D+Ftsu (Hydro)
168	440.8	92	70	1.2D+Ftsu+0.5L (Impact)
168	313.2	92	70	0.9D+Ftsu (Impact)
Floor 3				
175	367.4	24	24	1.2D+Ftsu+0.5L (Hydro)
175	261	24	24	0.9D+Ftsu (Hydro)
167	367.4	92	69	1.2D+Ftsu+0.5L (Impact)
167	261	92	69	0.9D+Ftsu (Impact)
Floor 4				
41	293.9	6	6	1.2D+Ftsu+0.5L (Hydro)
41	208.8	6	6	0.9D+Ftsu (Hydro)
94	293.9	92	15	1.2D+Ftsu+0.5L (Impact)
94	208.8	92	15	0.9D+Ftsu (Impact)
Floor 5				
10	220.4	1	1	1.2D+Ftsu+0.5L (Hydro)
10	156.6	1	1	0.9D+Ftsu (Hydro)
31	220.4	3	3	1.2D+Ftsu+0.5L (Impact)
31	156.6	3	3	0.9D+Ftsu (Impact)
Floor 6				
2	146.9	0	0	1.2D+Ftsu+0.5L (Hydro)
2	104.4	0	0	0.9D+Ftsu (Hydro)
7	146.9	1	1	1.2D+Ftsu+0.5L (Impact)
7	104.4	1	1	0.9D+Ftsu (Impact)
Floor 7				
1	73.5	0	0	1.2D+Ftsu+0.5L (Hydro)
1	52.2	0	0	0.9D+Ftsu (Hydro)
2	73.5	0	0	1.2D+Ftsu+0.5L (Impact)
2	52.2	0	0	0.9D+Ftsu (Impact)

A.15.1.2 Combined Gravity and Tsunami Loads

The column chosen at Grid Intersection A-3 from **Figure A-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure A-75 to **Figure A-77** shows the interaction diagram for the typical exterior column including the tsunami load combinations.

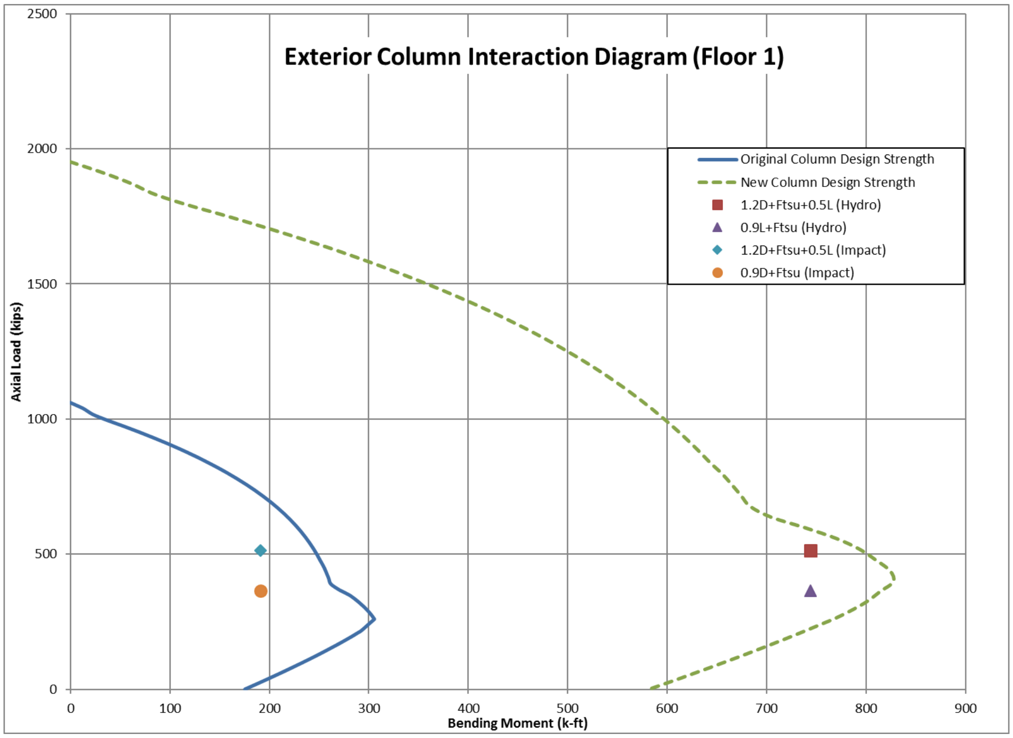

Figure A-75: Interaction diagram for typical ground floor exterior column showing tsunami load combinations

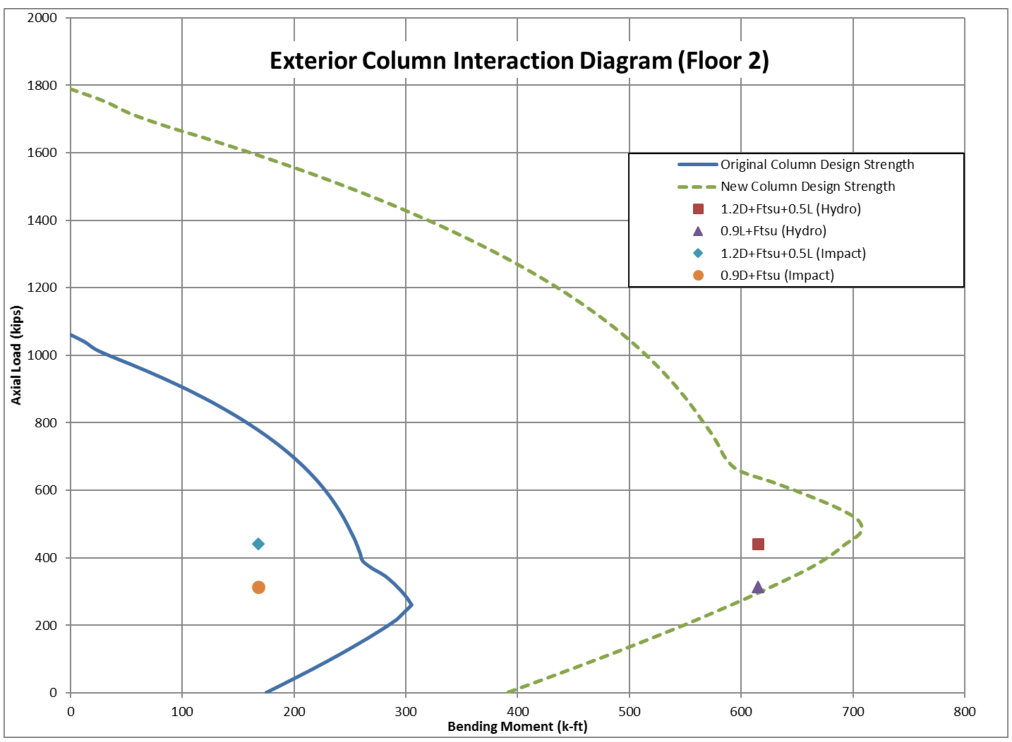

Figure A-76: Interaction diagram for typical 2nd floor exterior column showing tsunami load combinations

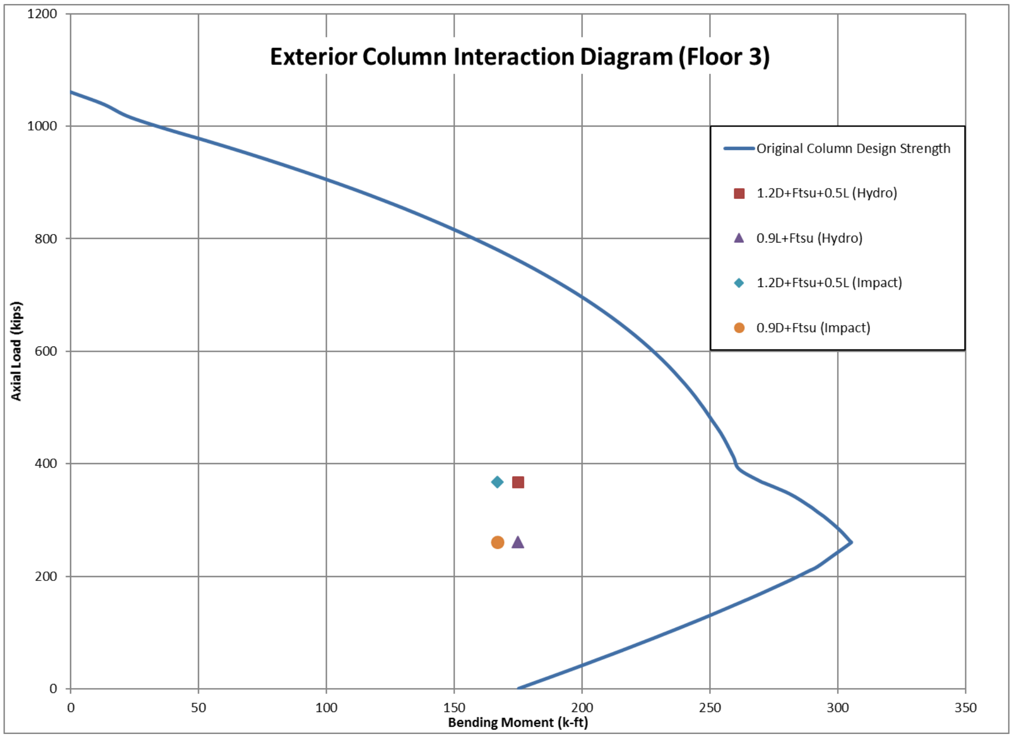

Figure A-77: Interaction diagram for typical 3rd floor exterior column showing tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

A.15.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure A-75** to **Figure A-77** the original exterior columns are adequate for log impact load, but the columns at the ground and 2nd floors must be strengthened to resist bending due to the hydrodynamic loads. Revised columns designs were developed to satisfy the hydrodynamic loads as shown in in **Figure A-78** to **Figure A-81**. The interaction diagrams for these new columns are shown in **Figure A-75** to **Figure A-76**.

Floor 1

End Section (A)

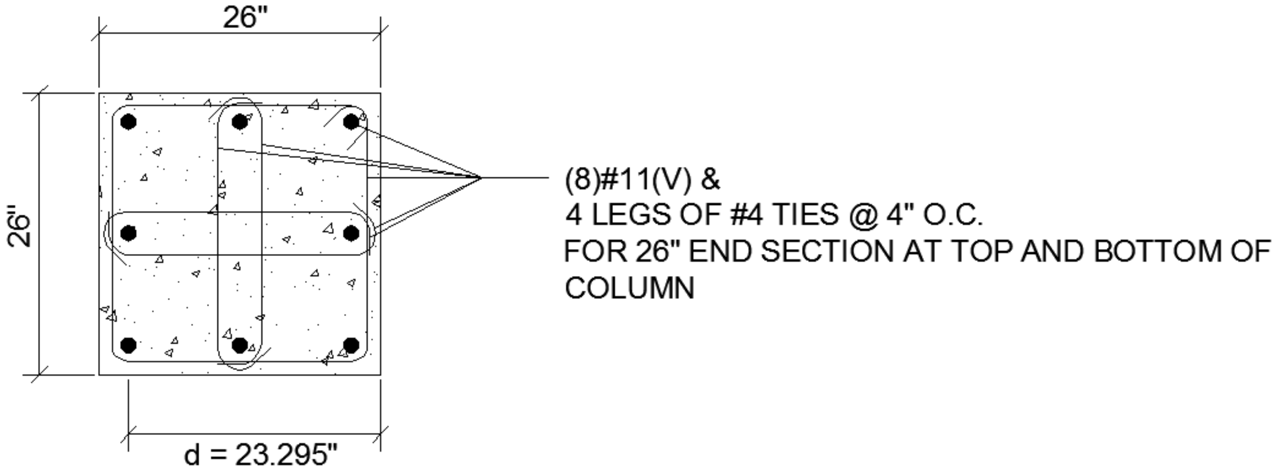

Figure A-78: Exterior column, cross-section at end of column at ground floor level based on tsunami design requirements.

Center Section (B)

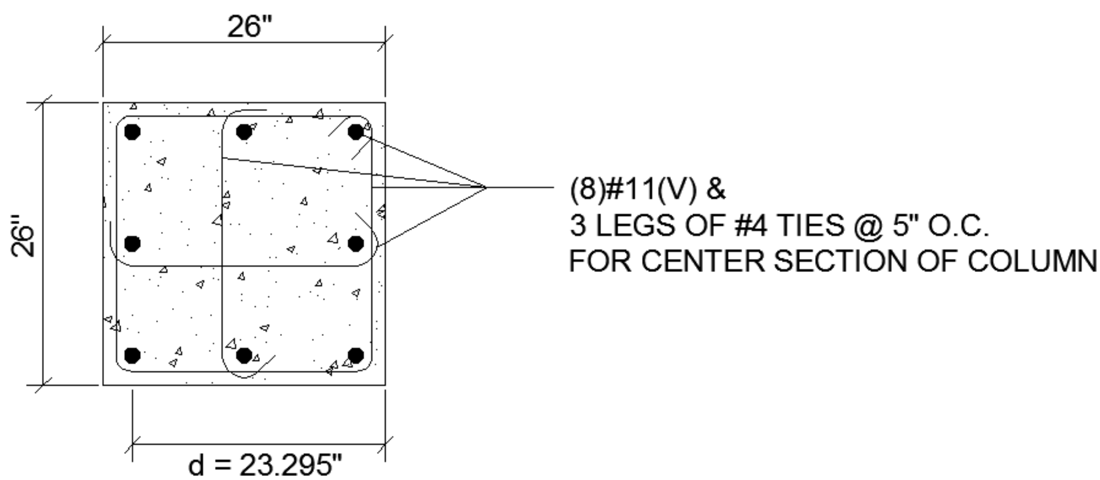

Figure A-79: Exterior column, cross-section at center of column at ground floor level based on tsunami design requirements.

Floor 2

End Section (A)

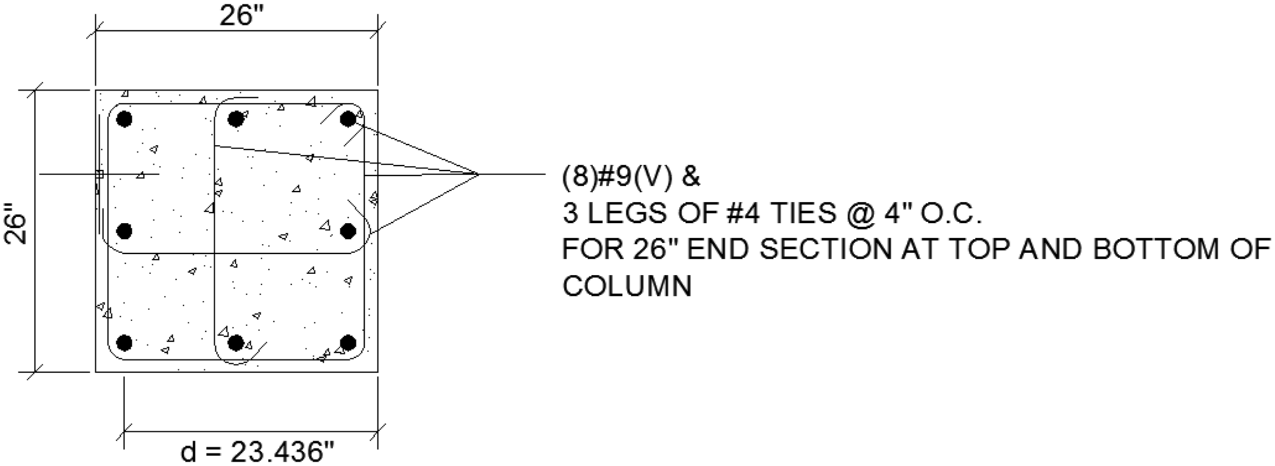

Figure A-80: Exterior column, cross section at end of column at the 2nd floor level based on tsunami design requirements.

Center Section (B)

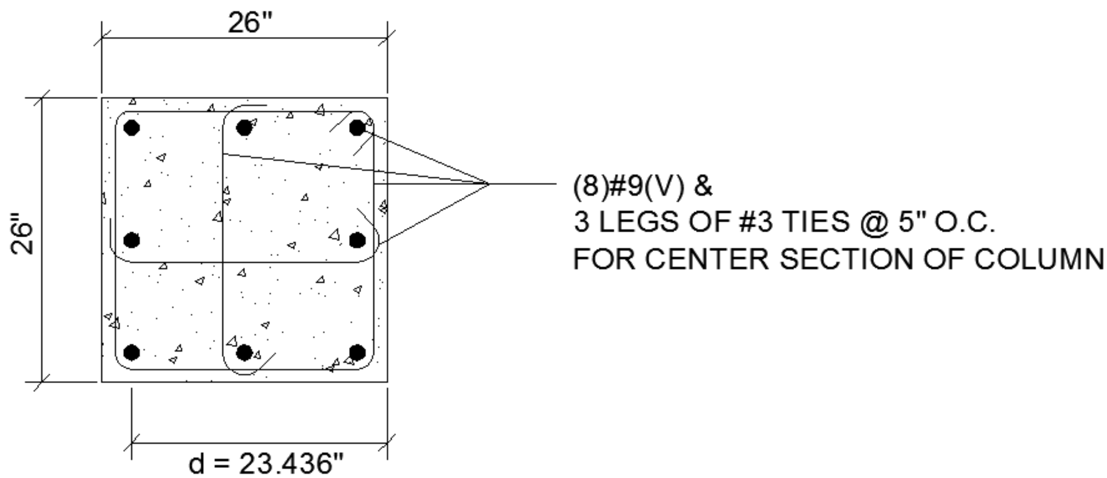

Figure A-81: Exterior column, cross-section at center of column at the 2nd floor level based on tsunami design requirements.

A.15.1.4 Exterior Column Shear Design

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = 365.4 kips.

The shear capacities of the 26”x26” columns with 4 leg #4 Stirrups at 4” o.c. in the end section and 4 leg #3 Stirrups at 5” o.c. in the center section are given by:

$$\varnothing V_n = \varnothing(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{365{,}400}{2{,}000\times26\times26}\right)26\times23.295/1{,}000$ = 97 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times23.295}{4\times1{,}000}$ = 280 kips

$$V_S = \frac{A_v f_y d}{s} = 280 \text{ kips} \ngtr 8\sqrt{f_C{}'}\text{ b d} = 8\times\sqrt{4{,}000}\times26\times23.295 = 306\text{ kips} \therefore \text{use } 280\text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times23.295}{5\times1{,}000}$ = 168 kips.

$$V_S = \frac{A_v f_y d}{s} = 168 \text{ kips} \ngtr 8\sqrt{f_C{}'}\text{ b d} = 8\times\sqrt{4{,}000}\times26\times23.295 = 306\text{ kips} \therefore \text{use } 168\text{ kips}$$

Therefore in the end sections, $ØV_n$ = 0.75 (97 + 280) = 283 k

and in the center sections, $ØV_n$ = 0.75 (97 + 168) = 199 k

At d: V_u = 282 k < ϕV_n = 283 k, therefore the column is adequate for shear at the end sections.

At d + h_c*:* V_u = 179 k < ϕV_n = 199 k, therefore the column is adequate for shear at the center section.

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 313.2 kips.

The shear capacities of the 26”x26” columns with 3 leg #4 Stirrups at 4” o.c. in the end section and 3 leg #3 Stirrups at 5” o.c. in the center section are given by:

$$ØV_n = Ø(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{313{,}200}{2{,}000\times26\times26}\right)26\times23.436/1{,}000$ = 95 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times23.436}{4\times1{,}000}$ = 211 kips

$$V_S = \frac{A_v f_y d}{s} = 211 \text{ kips} \ngtr 8\sqrt{f_C{}'}\text{ b d} = 8\times\sqrt{4{,}000}\times26\times23.463 = 308\text{ kips} \therefore \text{use } 211\text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times23.436}{5\times1{,}000}$ = 93 kips.

$$V_S = \frac{A_v f_y d}{s} = 93 \text{ kips} \ngtr 8\sqrt{f_C{}'}\text{ b d} = 8\times\sqrt{4{,}000}\times26\times23.463 = 308\text{ kips} \therefore \text{use } 93\text{ kips}$$

Therefore in the end sections, $ØV_n$ = 0.75 (95 + 211) = 229 k

and in the center sections, $ØV_n$ = 0.75 (95 + 93) = 141 k

At d: V_u = 217 k < ϕV_n = 229 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: V_u = 113 k < ϕV_n = 141 k, therefore the column is adequate for shear at the center section

Critical Shears in Columns at 3rd Floor:

At the critical axial load combination of (0.9*D* + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 261 kips.

The shear capacities of the 20"x20" columns with 3 leg #4 Stirrups at 4" o.c. in the end section and 3 leg #3 Stirrups at 5" o.c. in the center section are given by:

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{313{,}200}{2{,}000\times20\times20}\right)20 \times 17.5625/1{,}000$ = 59 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times17.5625}{4\times1{,}000}$ = 158 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times17.5625}{5\times1{,}000}$ = 69 kips.

Therefore in the end sections, $\text{Ø}V_n$ = 0.75 (59 + 158) = 163 k

and in the center sections, $\text{Ø}V_n$ = 0.75 (59 + 70) = 96 k

At d: V_u = 92 k < ϕV_n = 163 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: V_u = 69 k < ϕV_n = 96 k, therefore the column is adequate for shear at the center section

By inspection the remaining columns are adequate to resist the tsunami shear force.

Instead of the equivalent static load analysis performed above, it is permissible to use a non-linear analysis following the provisions of ASCE 41, or to perform a non-linear dynamic analysis of the column subjected to the debris impact strike.

A.15.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure A-16**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure A-82** and **Figure A-83** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the ends of the column, where *d* = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "d + h_c" from each end of the column, where d + h_c = 17.5 + 20 = 37.5 in.

Floor 1 – 7

End Section (A)

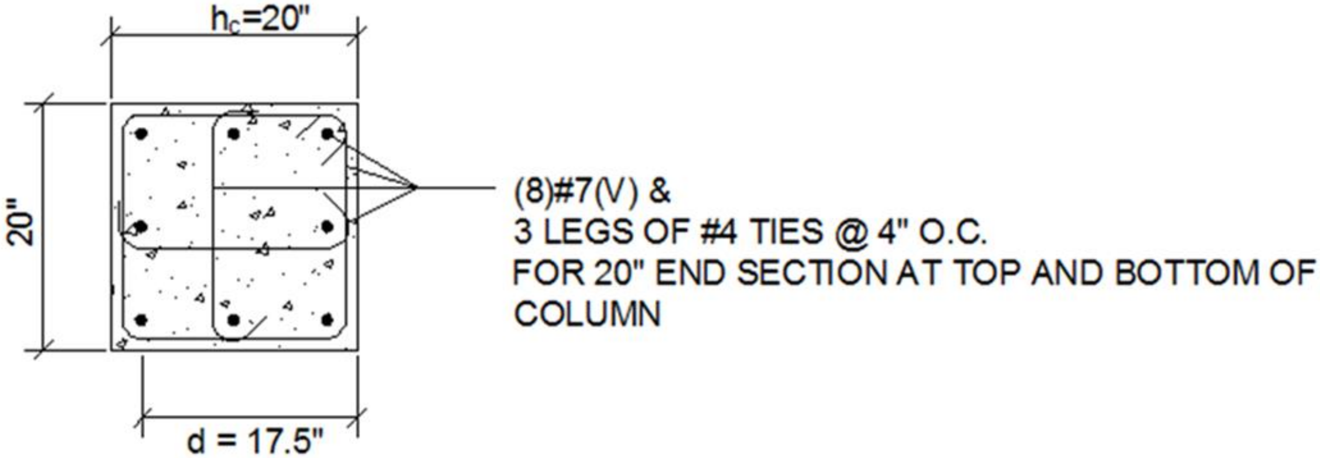

Figure A-82: Interior column, cross-section end of column at all floor levels based on SDC D design.

Center Section (B)

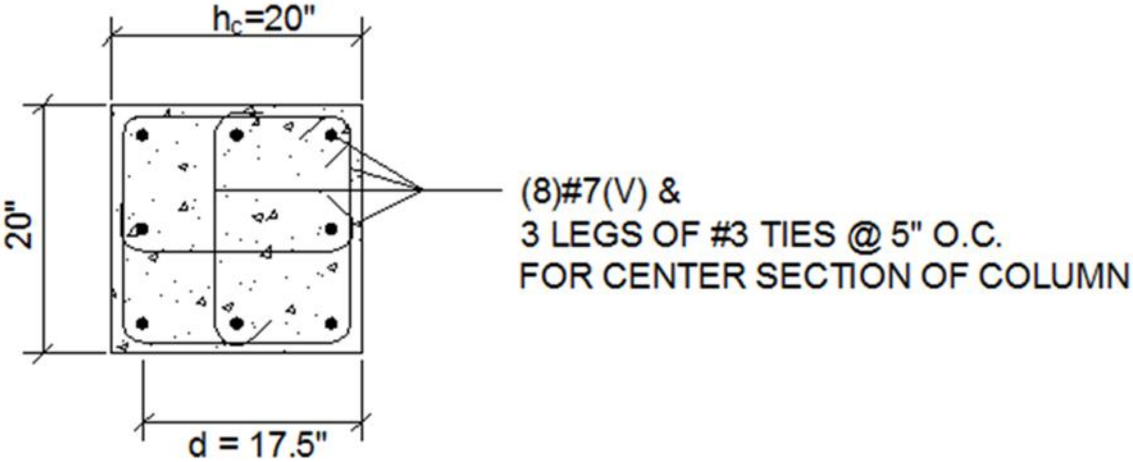

Figure A-83 Interior column, cross-section at center of column at all floor levels based on SDC D design.

A.15.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 17.83 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$$P_D = [128(14.58)(28)(6)+ 123(17.83)(28)+1.67^2(150)(66)]/1000 = 472\ k$$

Floor Live load reduction factor = $0.25+15/[4(17.83)(28)(6)]^{0.5} = 0.487$, therefore, column base live load is:

$P_L = 0.487[55(17.83)(28)(6)]/1000 = 80.2\ k$

Roof Live Load reduction factor = $R_1R_2 = [1.2\text{-}(0.001)(17.83)(28)](1.0) = 0.701$*, column roof live load is:*

$P_{Lr} = 0.701(20)(17.83)(28)/1000 = 6.61\ k$

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

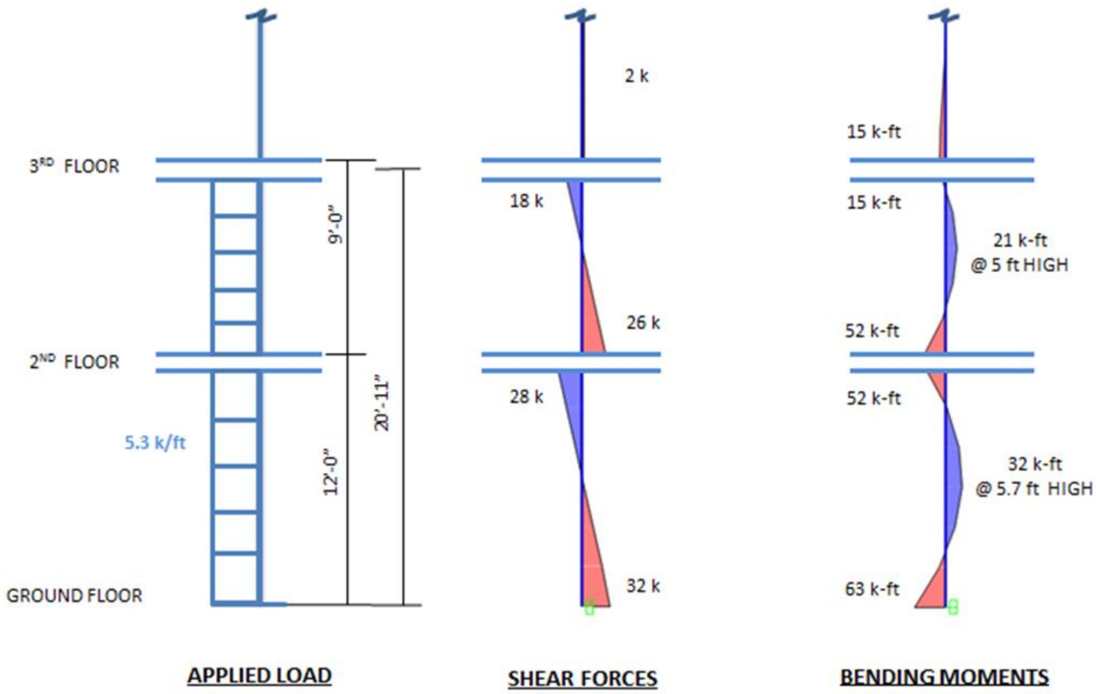

Figure A-84: Hydrodynamic loading on interior column of Seaside residential building due to Load Case 2

Table A-7 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table A-7: Results from loading conditions of Seaside residential building interior column

Moment	Axial Load	Shear @ d	Shear @ d + h_c	Load Combination
k-ft	Kips	Kips	Kips	
Floor 1				
63	811.8	24	15	1.2D+Ftsu+0.5L (Hydro)
63	566.1	24	15	0.9D+Ftsu (Hydro)
Floor 2				
52	676.5	18	10	1.2D+Ftsu+0.5L (Hydro)
52	471.8	18	10	0.9D+Ftsu (Hydro)
Floor 3				
15	541.2	2	2	1.2D+Ftsu+0.5L (Hydro)
15	377.4	2	2	0.9D+Ftsu (Hydro)
Floor 4				
3	405.9	0.5	0.5	1.2D+Ftsu+0.5L (Hydro)
3	283.1	0.5	0.5	0.9D+Ftsu (Hydro)
Floor 5				
1	270.6	0	0	1.2D+Ftsu+0.5L (Hydro)
1	188.7	0	0	0.9D+Ftsu (Hydro)
Floor 6				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.4	0	0	0.9D+Ftsu (Hydro)
Floor 7				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.4	0	0	0.9D+Ftsu (Hydro)

A.15.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure A-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure A-85 shows the interaction diagram for the typical exterior column including the tsunami load combinations.

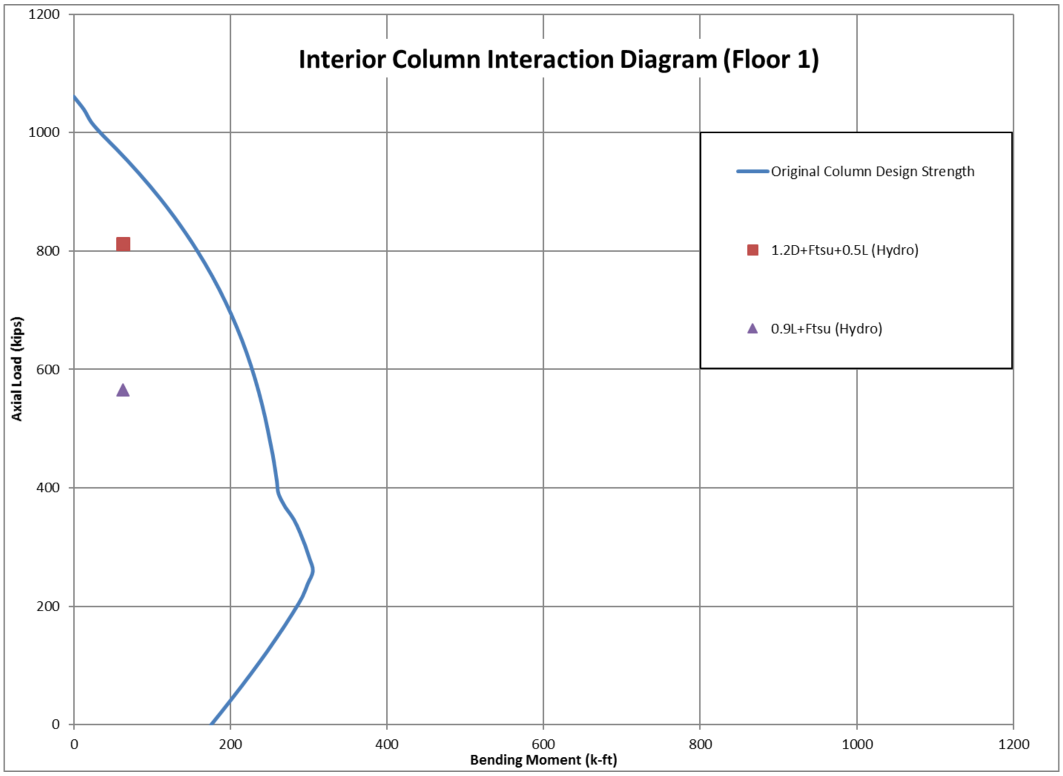

Figure A-85: Interaction diagram for typical ground floor residential interior column showing tsunami load combinations

A.15.2.1 Interior Column Shear Design

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of (1.2D + F_{TSU} + 0.5L) per **Eqn. 6.8.3.3.-1a,** P_u = 811.8 kips.

The shear capacities of the existing 20"x20" column with 3 leg #4 Stirrups @ 4" o.c. in the end sections and 3 leg #3 Stirrups @ 5" o.c. in the center section are given by:

$\emptyset V_n = \emptyset(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{811{,}800}{2{,}000\times20\times20}\right)20 \times 17.5625/1{,}000$ = 90 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times17.5625}{4\times1{,}000}$ = 158 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times17.5625}{5\times1{,}000}$ = 70 kips

Therefore in the end sections, $\emptyset V_n$ = 0.75 (90 + 158) = 186 k

and in the center sections, $\emptyset V_n$ = 0.75 (90 + 70) = 120 k

At d: V_u = 37 k < ϕV_n = 186 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 24 k < ϕV_n = 120 k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear force.

A.15.3 Typical Exterior Wall Design

A section of exterior wall along Grid Line D from **Figure A-16** adjacent to the mechanical room was analyzed. The wall is part of the lateral resisting system for seismic loads, acting as a shear wall for longitudinal forces and boundary element for transverse forces. Seismic Design Category D design and detailing of the 10" thick wall resulted in the reinforcement layout shown in **Figure A-86** to **Figure A-88**. The wall will now be checked for tsunami loads.

For comparative purposes with the debris impact loads, the ultimate shear forces and bending moments are provided for an effective width of wall equal to 5.67 ft. The critical shear force occurs at a distance "*d*" from the base of the wall, where *d* = 10-0.75-1"/2 = 8.75 in.

Floor 1

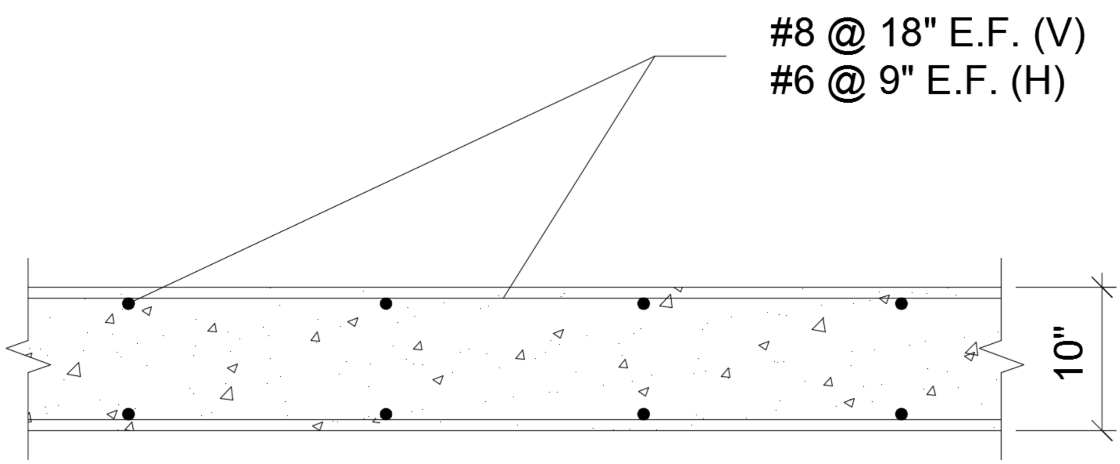

Figure A-86: Segment of exterior wall cross-section at the ground floor level based on SDC D design.

Floor 2

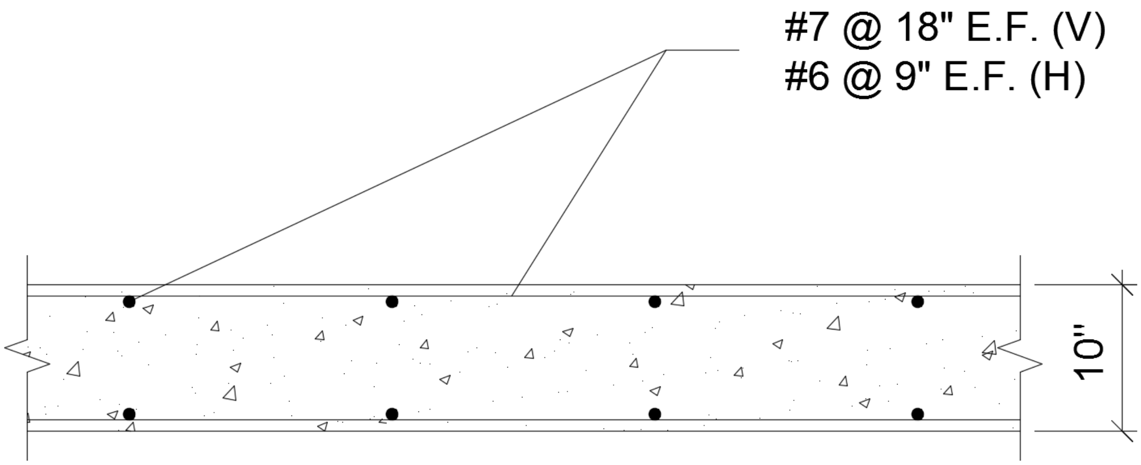

Figure A-87: Segment of exterior wall cross-section at the 2nd floor level based on SDC D design.

Floor 3 – 7

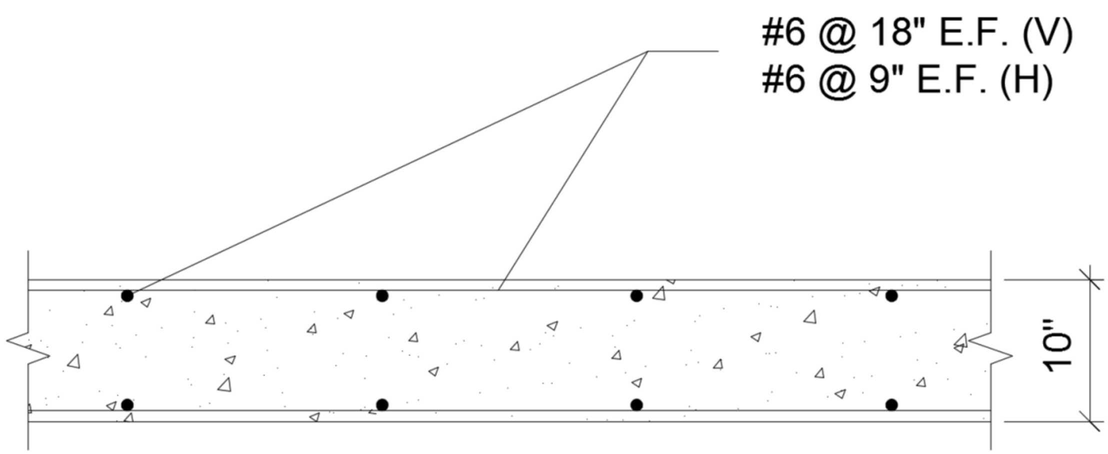

Figure A-88: Segment of exterior wall cross-section at the 3rd – 7th floor level based on SDC D design.

A.15.3.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 5.5 ft. Gravity loads will be computed per foot width of wall.

The Dead Load at the base of the wall is:

$P_D = [128(5.5)(1)(6)+123(5.5)(1)+0.83(1)(150)(66)]/1000 = 13.1\ k/ft$

Floor Live load reduction: Reduction Factor = $0.25+15/[1(5.5)(28)(6)]^{0.5} = 0.743$

$P_L = 0.743[55(5.5)(1)(6)]/1000 = 1.35\ k/ft$

Roof Live Load reduction: Reduction Factor = $R_1R_2= [1.2-(0.001)(5.5)(28)](1.0) = 1.05$*, Use 1.0*

$P_{Lr} = 20(5.5)(1)/1000 = 0.110\ k/ft$

Analysis of a 5.67 foot width of wall with the applied tsunami hydrodynamic loads from Load Case 2 results in the following bending moment and shear force diagrams:

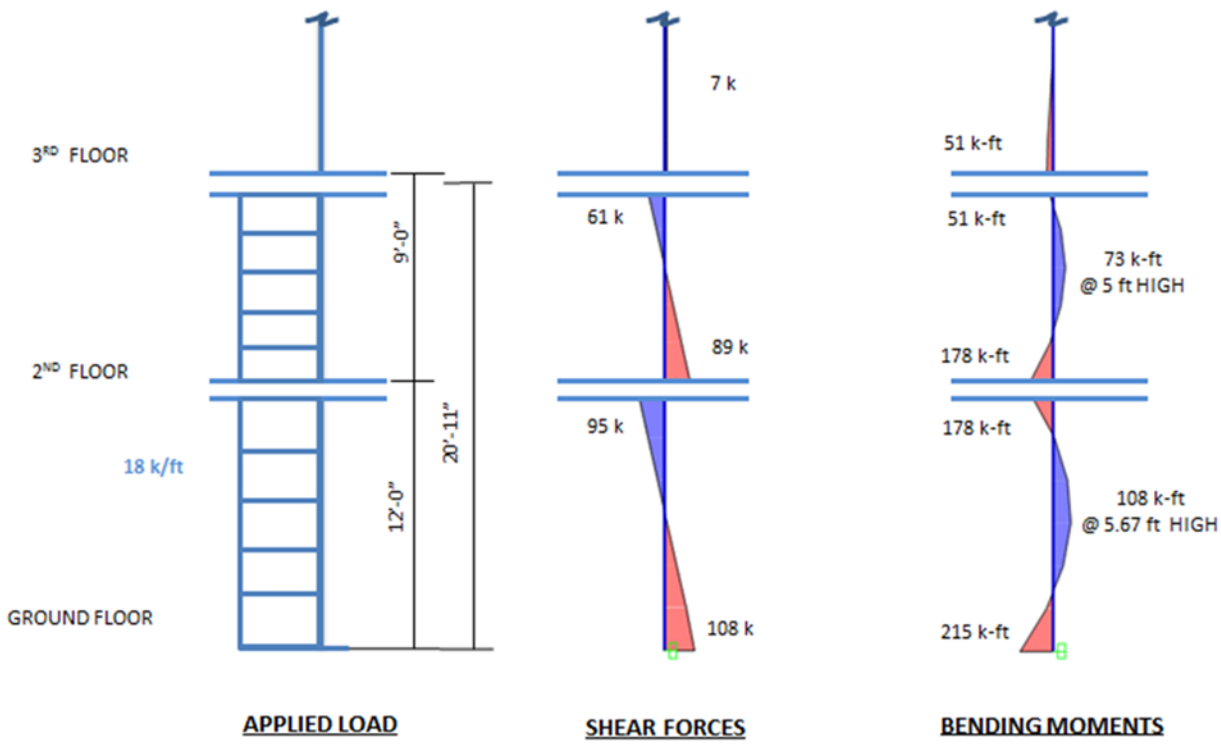

Figure A-89: Hydrodynamic loading on exterior wall of Seaside residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike, acts over an effective width of 5.67 ft, at a point just below the slab at each inundated floor for maximum shear and at the mid-height of the clear column height for maximum bending moments. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure A-90** to **Figure A-93**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown in **Figure A-94** to **Figure A-97**.

Point load at d:

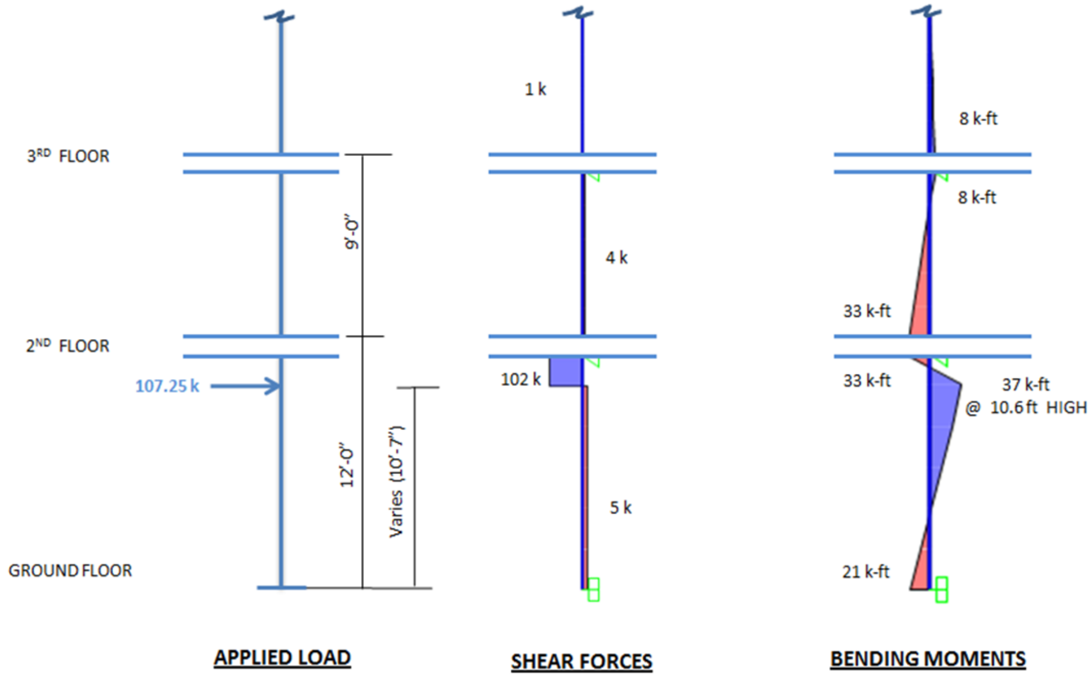

Figure A-90: Impact load applied at d away from the end of column on the ground floor

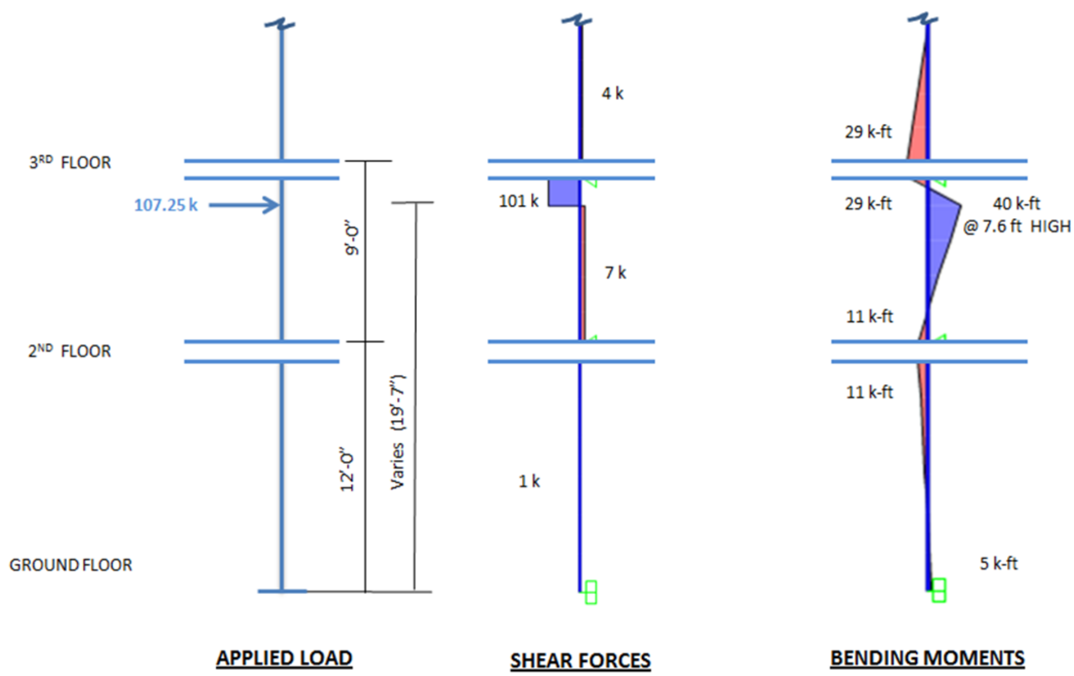

Figure A-91: Impact load applied at d away from the end of column on the 2nd floor

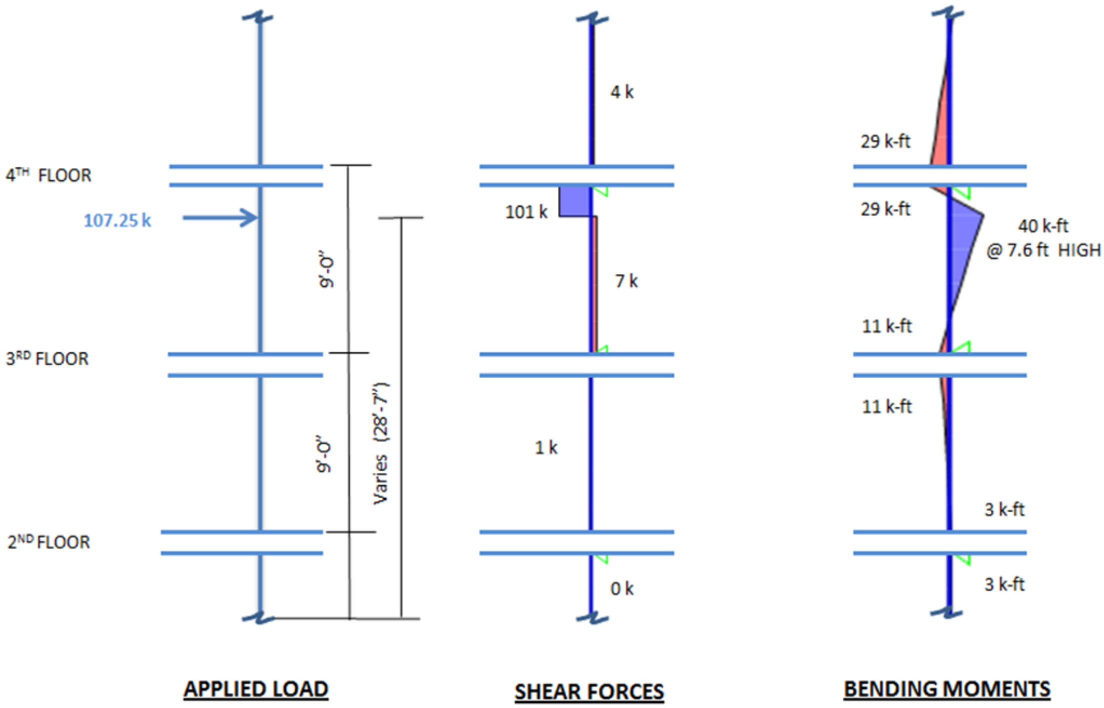

Figure A-92: Impact load applied at d away from the end of column on the 3rd floor

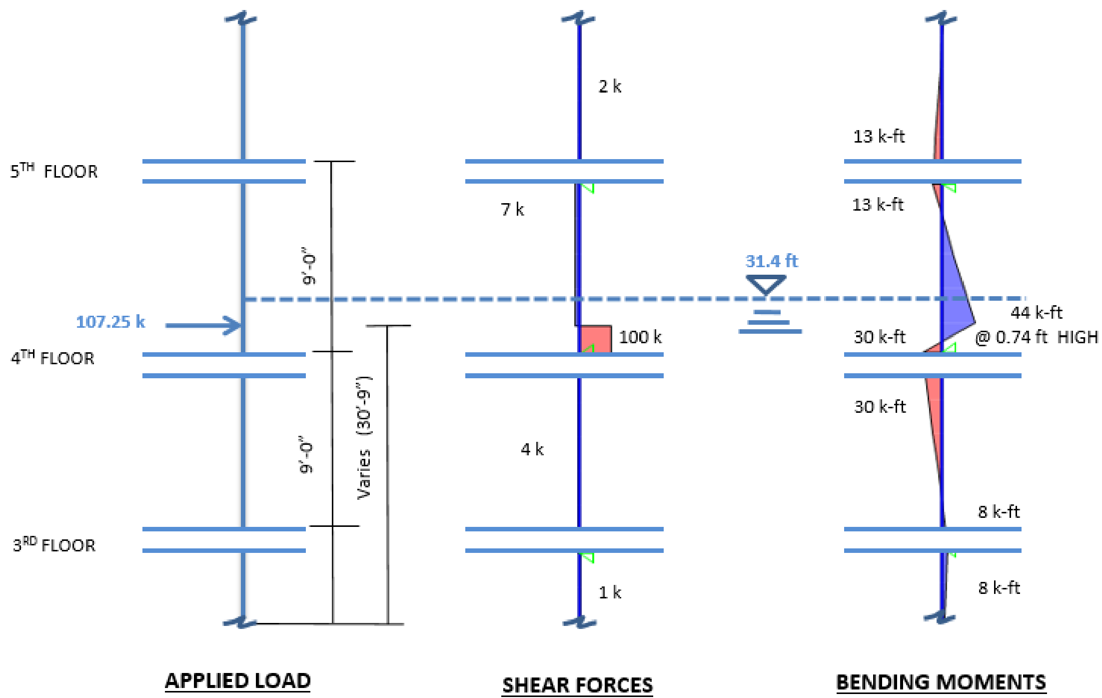

Figure A-93: Impact load applied at d away from the end of column on the 4th floor

Point load at Mid-heights:

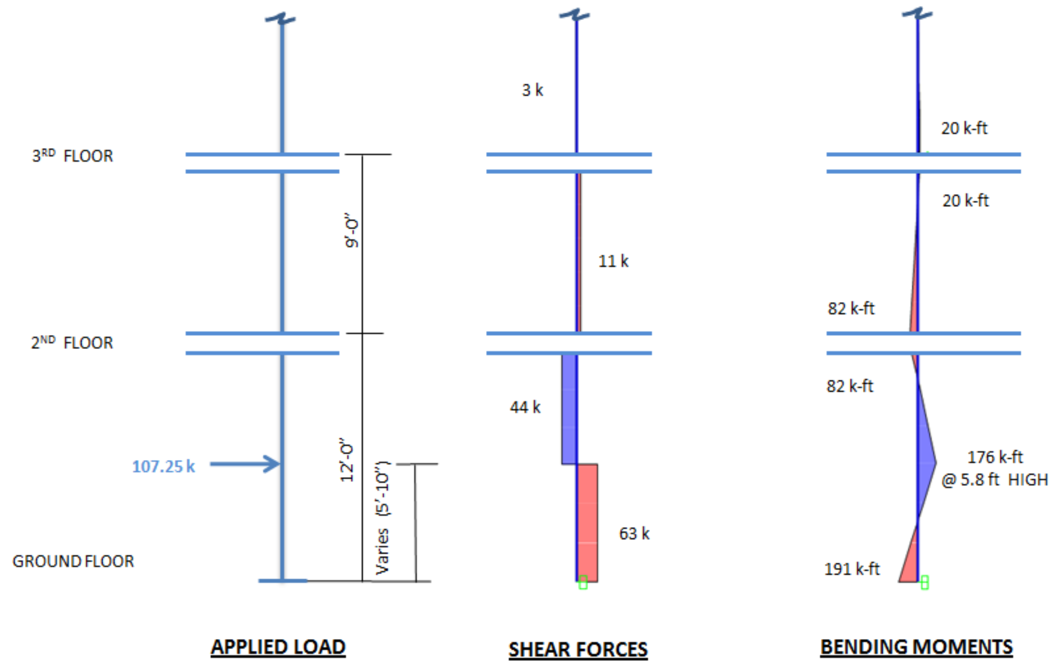

Figure A-94: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

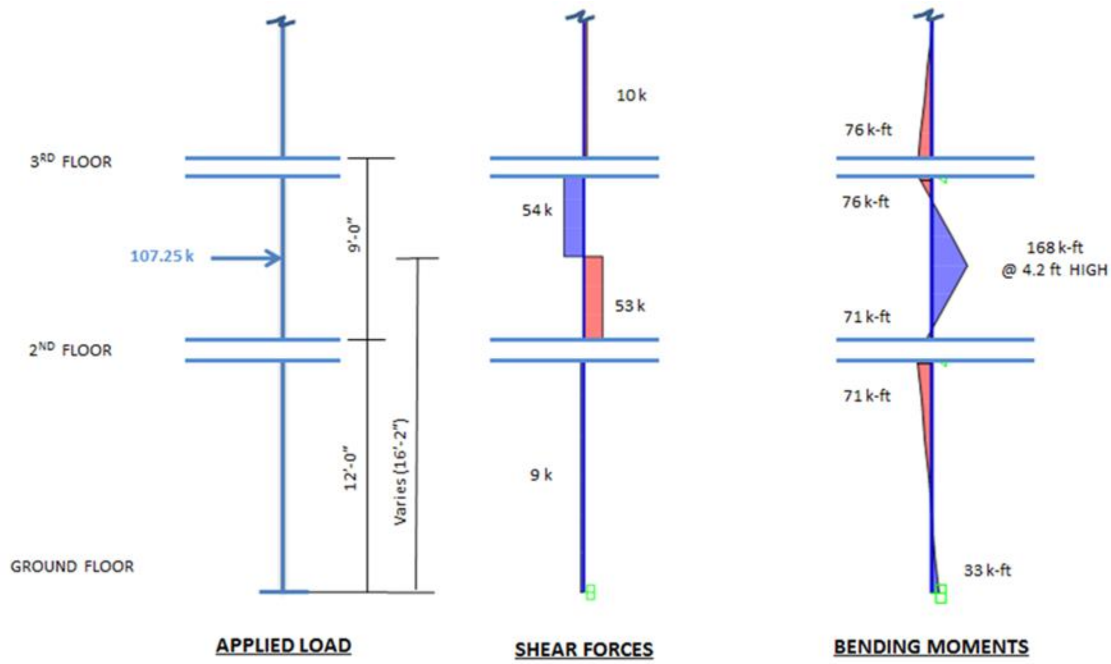

Figure A-95: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

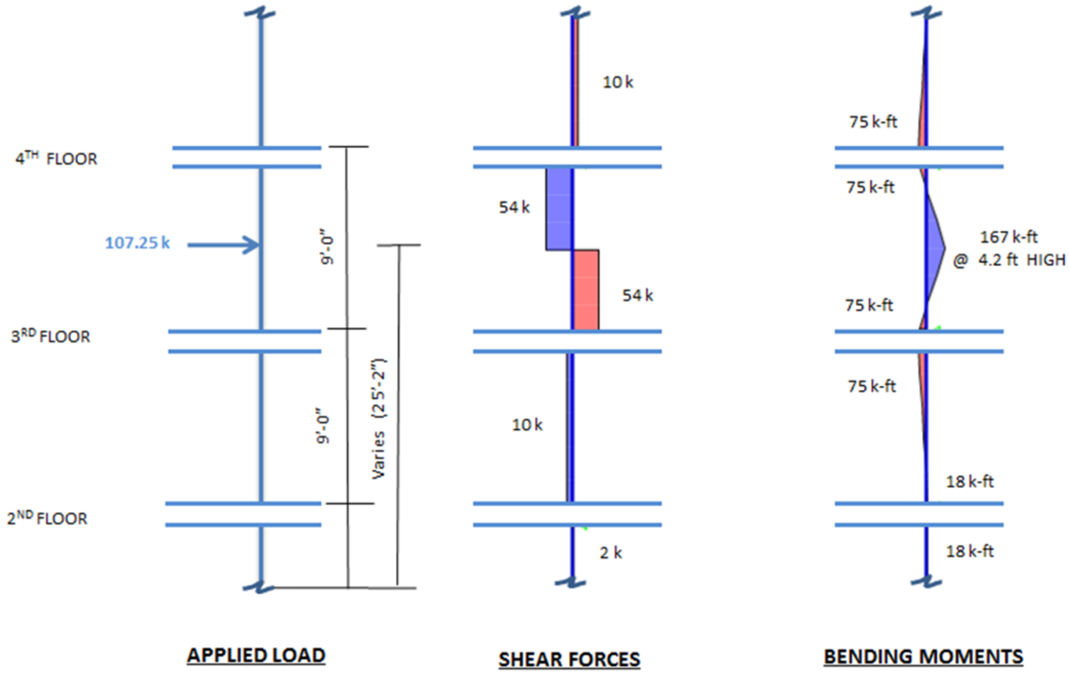

Figure A-96: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

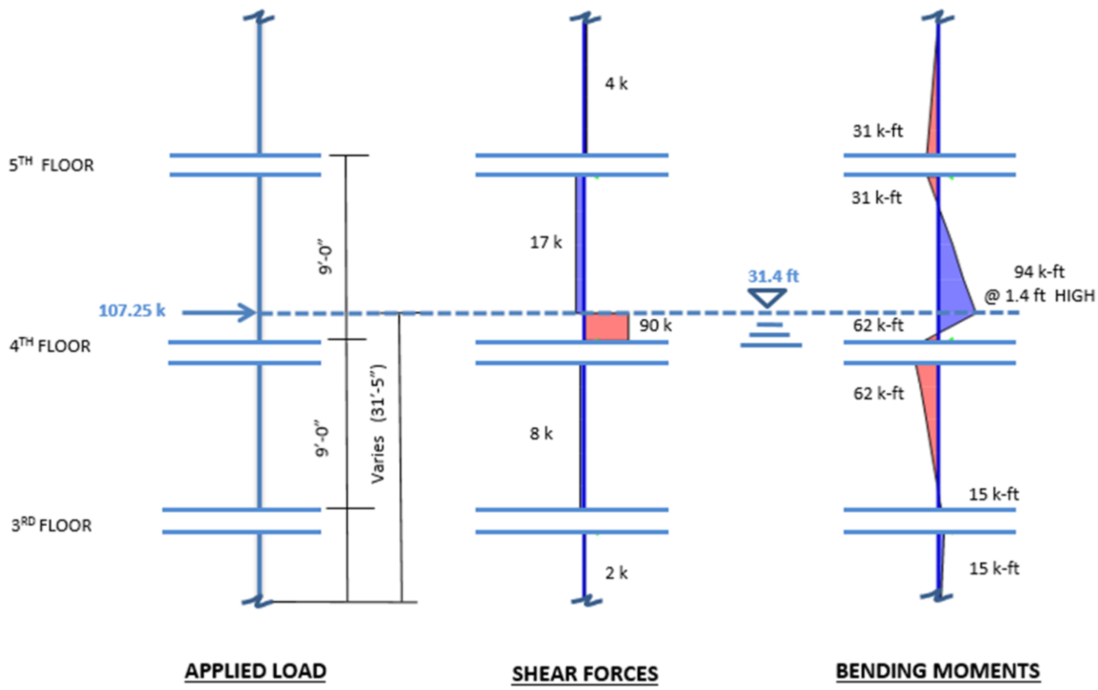

Figure A-97: Impact load applied at 1.4 ft away from the assumed lateral restraint instead of the mid-height of the assumed lateral restraint points at top and bottom of the 4th floor column

Table A-8: Results from loading conditions of Seaside residential building exterior shear wall

Moment K-ft	Axial Load Kips	Shear @ d Kips	Load combination
Floor 1			
215	92.91	95	1.2D+Ftsu+0.5L (Hydro)
215	6.89	95	0.9D+Ftsu (Hydro)
176	92.91	102	1.2D+Ftsu+0.5L (Impact)
176	6.89	102	0.9D+Ftsu (Impact)
Floor 2			
178	79.63	76	1.2D+Ftsu+0.5L (Hydro)
178	5.90	76	0.9D+Ftsu (Hydro)
168	79.63	101	1.2D+Ftsu+0.5L (Impact)
168	5.90	101	0.9D+Ftsu (Impact)
Floor 3			
51	66.36	7	1.2D+Ftsu+0.5L (Hydro)
51	4.92	7	0.9D+Ftsu (Hydro)
167	66.36	101	1.2D+Ftsu+0.5L (Impact)
167	4.92	101	0.9D+Ftsu (Impact)
Floor 4			
12	53.09	2	1.2D+Ftsu+0.5L (Hydro)
12	3.93	2	0.9D+Ftsu (Hydro)
80	53.09	100	1.2D+Ftsu+0.5L (Impact)
80	3.93	100	0.9D+Ftsu (Impact)
Floor 5			
3	39.82	0	1.2D+Ftsu+0.5L (Hydro)
3	2.95	0	0.9D+Ftsu (Hydro)
25	39.82	2	1.2D+Ftsu+0.5L (Impact)
25	2.95	2	0.9D+Ftsu (Impact)
Floor 6			
1	26.54	0	1.2D+Ftsu+0.5L (Hydro)
1	1.97	0	0.9D+Ftsu (Hydro)
6	26.54	0	1.2D+Ftsu+0.5L (Impact)
6	1.97	0	0.9D+Ftsu (Impact)
Floor 7			
0	13.27	0	1.2D+Ftsu+0.5L (Hydro)
0	0.98	0	0.9D+Ftsu (Hydro)
1	13.27	0	1.2D+Ftsu+0.5L (Impact)
1	0.98	0	0.9D+Ftsu (Impact)

A.15.3.2 Existing Exterior Shear Wall Design for Combined Gravity and Tsunami Loads

The shear wall chosen at Grid Line D from **Figure A-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**.

Figure A-98 to **Figure A-101** shows the interaction diagram for the typical exterior shear wall including the tsunami load combinations.

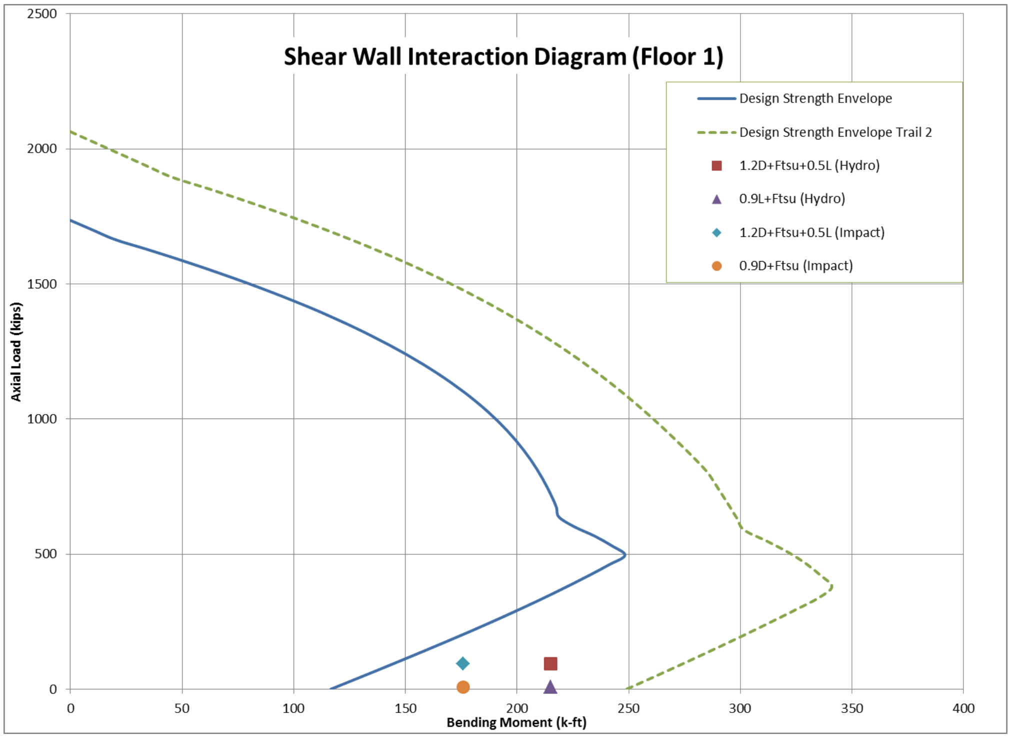

Figure A-98: Interaction diagram for typical ground floor exterior wall segment showing tsunami load combinations

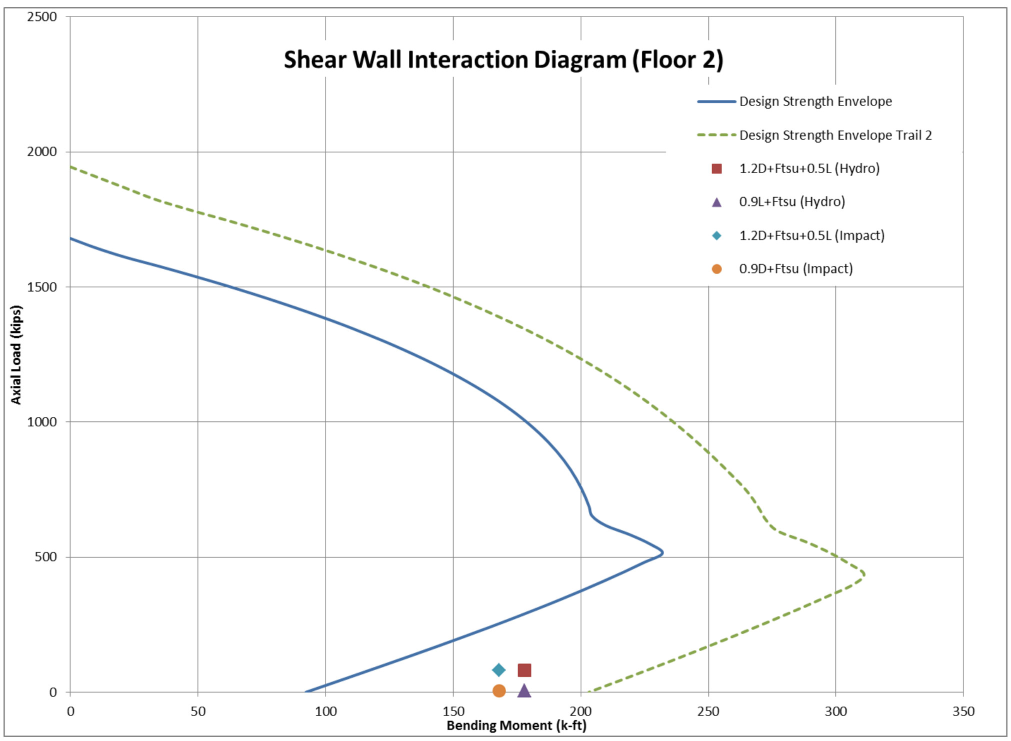

Figure A-99: Interaction diagram for typical 2nd floor exterior wall segment showing tsunami load combinations

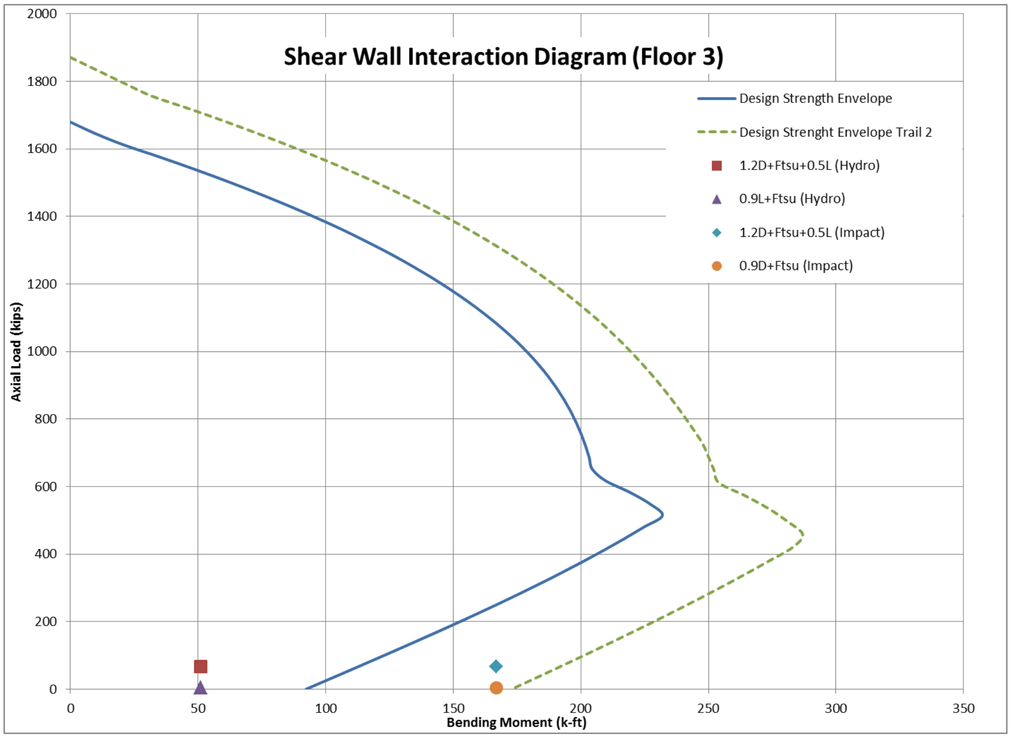

Figure A-100: Interaction diagram for typical 3rd floor exterior wall segment showing tsunami load combinations

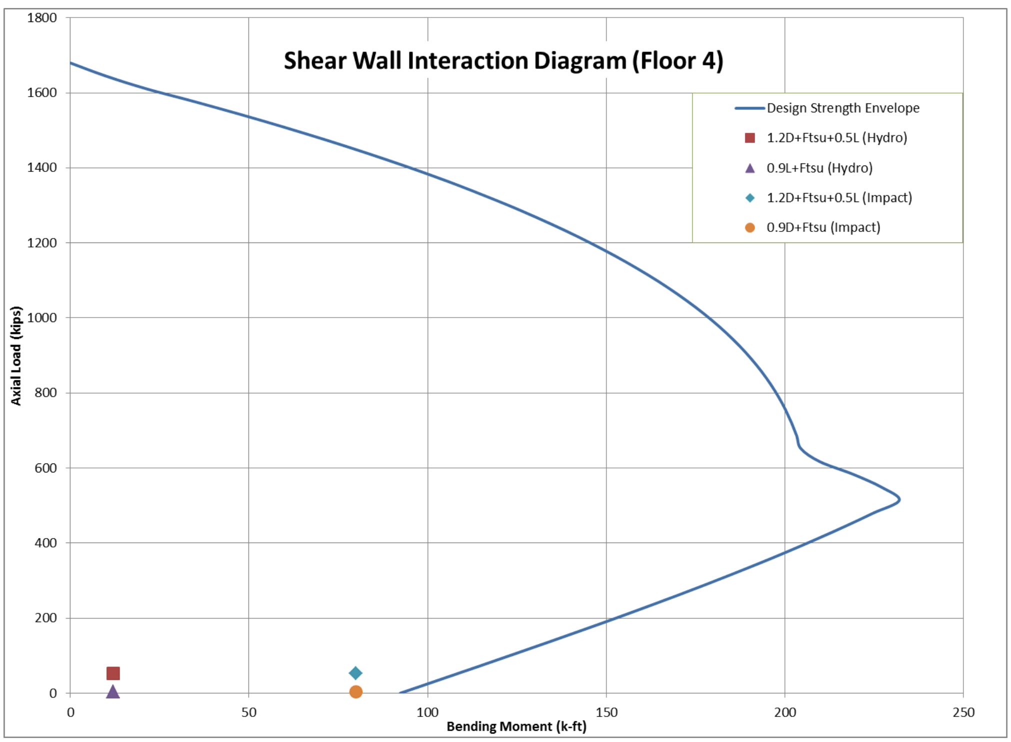

Figure A-101: Interaction diagram for typical 4th floor exterior wall segment showing tsunami load combinations

By inspection the remaining shear walls are adequate to resist the tsunami bending moments.

A.15.3.3 New Typical Shear Wall Design

The interaction diagrams show that all the walls are adequate for bending moments due to hydrodynamic load and derbies impact. Although the floors are adequate for the bending moments Floors 1 – 4 are not adequate for shear loading, **Figure A-102** to **Figure A-105** show the revised wall designs required to resist the tsunami loads. **Figure A-106** shows the side view of the wall with shear stud rails included.

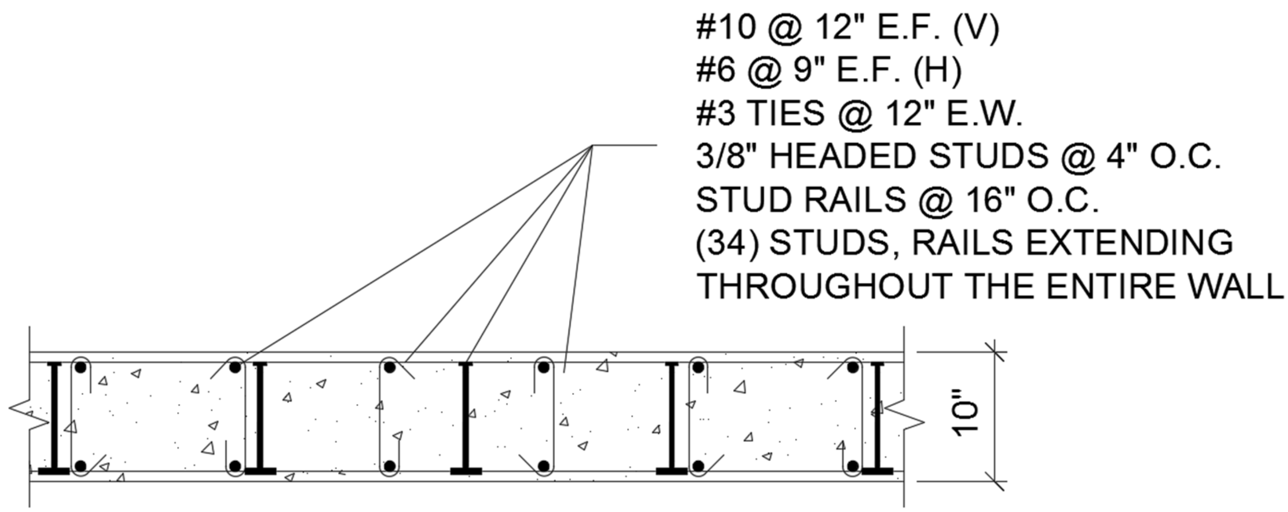

Figure A-102: New exterior wall, cross-section at the ground floor level based on tsunami design requirements.

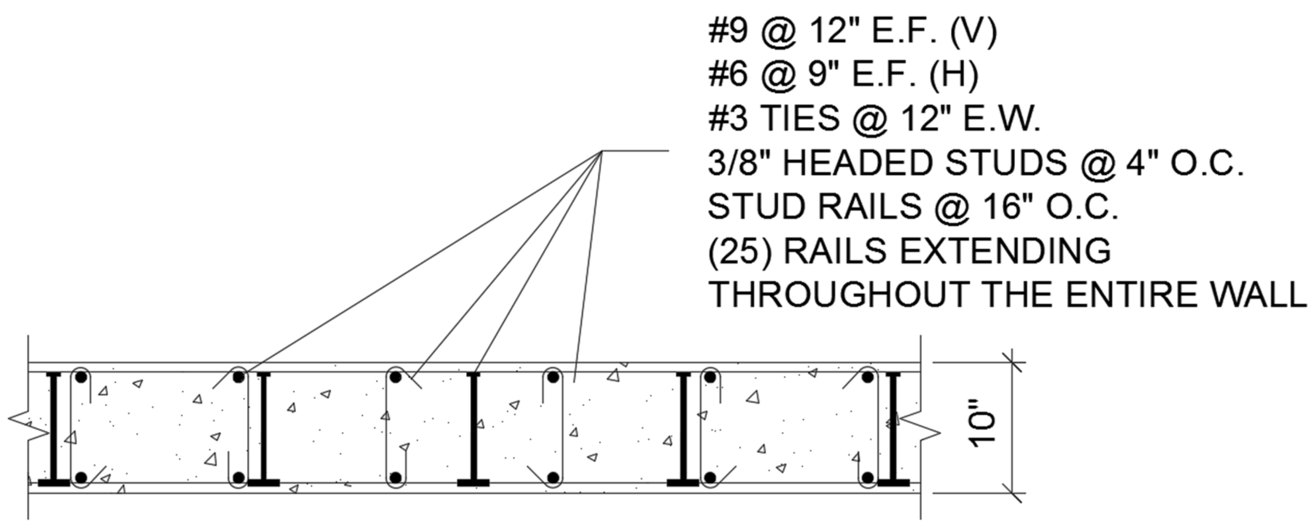

Figure A-103: New exterior wall, cross-section at the 2nd floor level based on tsunami design requirements.

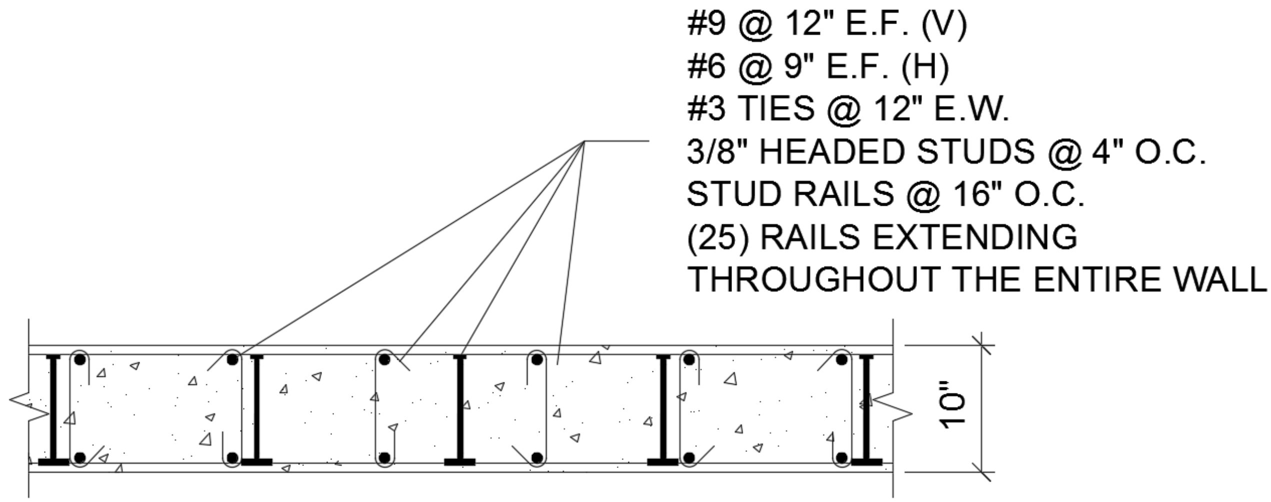

Figure A-104: New exterior wall, cross-section at the 3rd floor level based on tsunami design requirements.

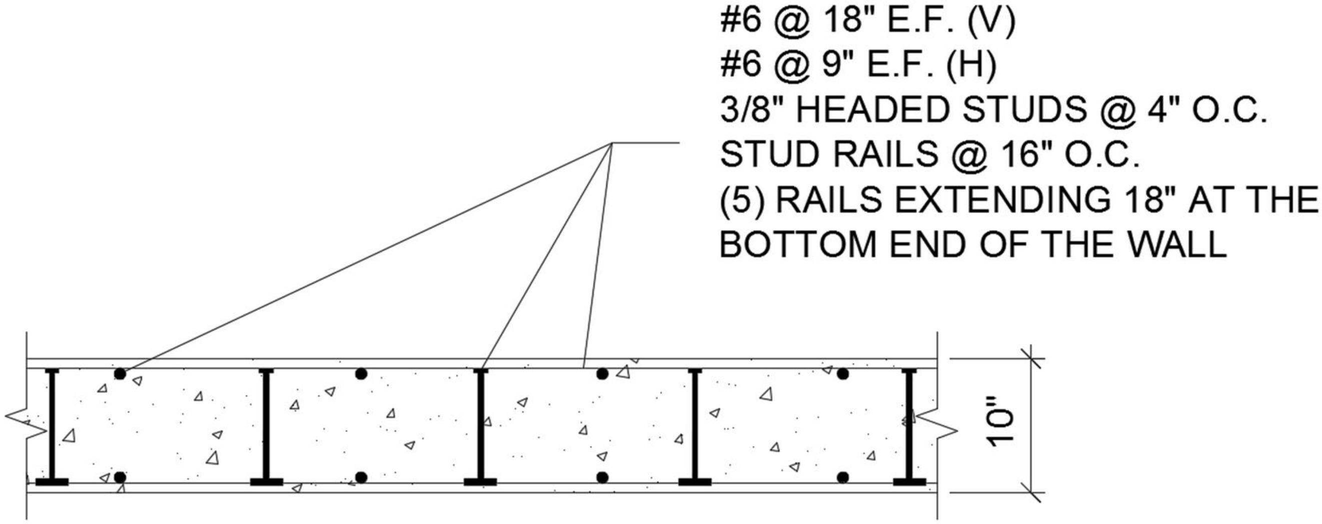

Figure A-105: New exterior wall, cross-section at the 4th floor level based on tsunami design requirements.

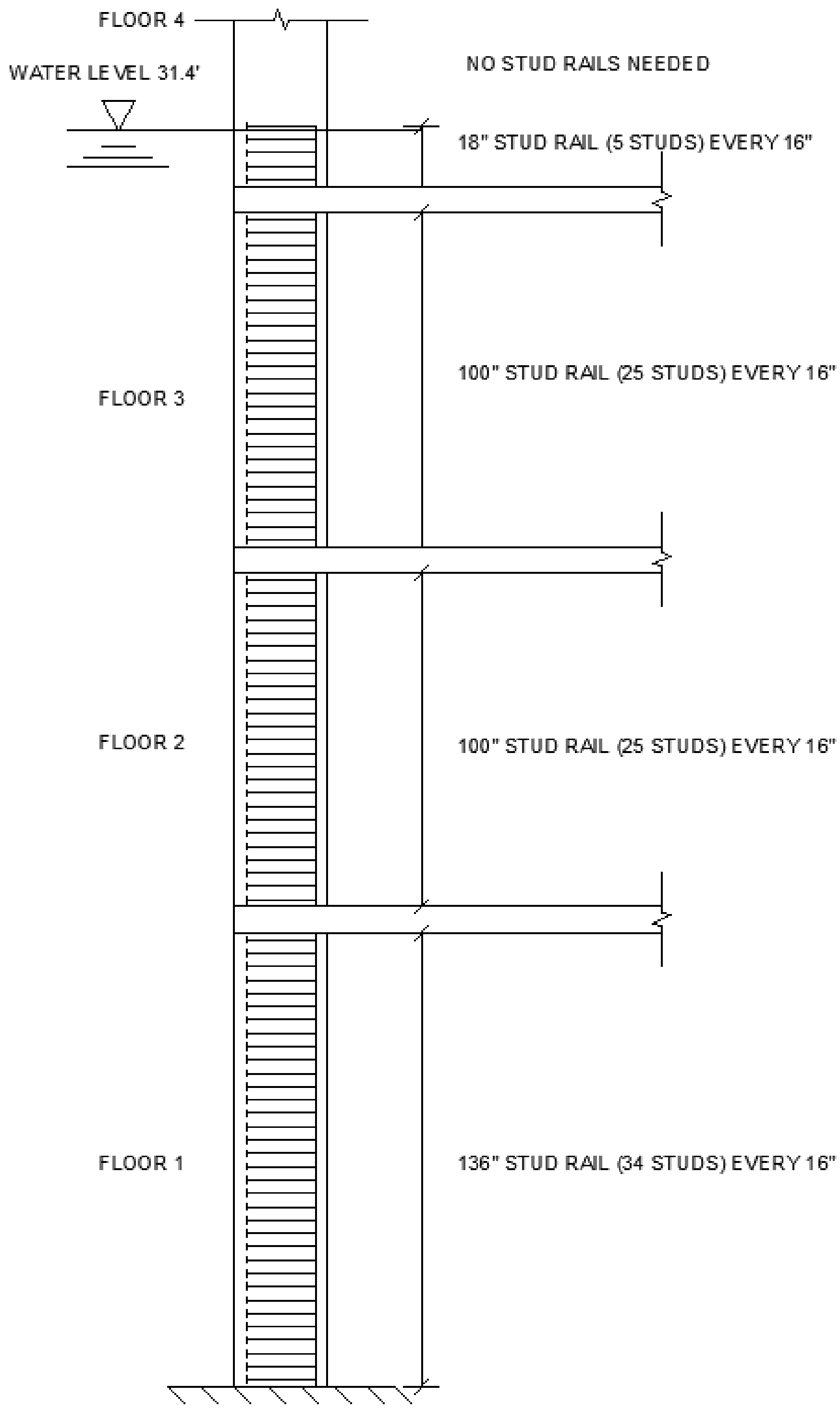

Figure A-106: Stud Rail Diagram for the Floor 1 – 4

A.15.3.4 Exterior Shear Wall Shear Design

Critical Shears:

1^{st} Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$), P_u = 6.89 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{807}{2{,}000\times 68\times 10}\right)\,1\,\sqrt{4000}\,\times 68\times 8.75/1{,}000$ = 75 kips

$N_u = \left(\frac{P_u\times 1{,}000}{I_w}\right)\times b = \left(\frac{6.89\times 1{,}000}{580}\right)\times 68 = 807\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (75+0) = 56 kips

V_{tsu} = 102 kips > $ØV_n$ = 56 kips ∴ 61 kips needed

Shear Capacity for New Shear Wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{807}{2{,}000\times 68\times 10}\right)\,1\,\sqrt{4000}\,\times 68\times 8.75/1{,}000$ = 75 kips

$N_u = \left(\frac{P_u\times 1{,}000}{I_w}\right)\times b = \left(\frac{6.89\times 1{,}000}{580}\right)\times 68 = 807\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (75+0) = 56 kips

V_{tsu} = 102 kips > $ØV_n$ = 56 kips ∴ 61 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n\times A_v\times f_y\times d}{s} = \frac{4.25\times 0.11\times 60\times 8.75}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"}$ = 4.25

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 = 8.75/2 = 4.375 in

$s_{needed} = \frac{n\times A_v\times f_y\times d}{V_{S\,(needed)}} = \frac{4.25\times 0.11\times 60\times 8.75}{61}$ = 4.1 in

$\therefore s_{used}$ = 4 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (75 + 62) = 103 Kips

$ØV_n$ = 103 Kips > V_{Tsu} = 102 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ mid\text{-}height}$ = 63 Kips > $ØV_c$ = 56 Therefore the rails go up the entire wall of the Shear Wall

2nd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 5.9 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{692}{2{,}000\times68\times10}\right)\ 1\sqrt{4000}\ \times 68\times 8.8125/1{,}000$ = 76 kips

$N_u = (\frac{P_u\times1{,}000}{I_w})\times b = \left(\frac{5.9\times1{,}000}{580}\right)\times 68 = 692\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 101 kips > $ØV_n$ = 57 kips $\therefore$ 59 kips needed

Shear Capacity for New Shear Wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{692}{2{,}000\times68\times10}\right)\ 1\sqrt{4000}\ \times 68\times 8.78125/1{,}000$ = 76 kips

$N_u = (\frac{P_u\times1{,}000}{I_w})\times b = \left(\frac{5.9\times1{,}000}{580}\right)\times 68 = 692\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 101 kips > $ØV_n$ = 57 kips $\therefore$ 59 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n\times A_v\times f_y\times d}{s} = \frac{4.25\times0.11\times60\times8.8125}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"} = 4.25$

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 = 8.8125/2 = 4.4 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{4.25 \times 0.11 \times 60 \times 8.75}{59}$ = 4.2 in

∴ s_{used} = 4 in

ØV$_n$ = Ø(V$_C$ + V$_S$) = 0.75 (76 + 62) = 103 Kips

ØV$_n$ = 103 Kips > V_{Tsu} = 101 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ mid\text{-}height}$ = 54 Kips < ØV$_c$ = 57 The rails go up the entire wall of the Shear Wall

3rd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 4.92 k:

Shear Capacity of existing shear wall (10" thick):

ØV$_n$ = Ø(V$_C$ + V$_S$)

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{577}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{4.92 \times 1{,}000}{580}\right) \times 68 = 577\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

ØV$_n$ = Ø(V$_C$ + V$_S$) = 0.75 (76+0) = 57 kips

V_{tsu} = 101 kips > ØV$_n$ = 57 kips ∴ 58 kips needed

Shear Capacity for New Shear Wall (10" thick):

ØV$_n$ = Ø(V$_C$ + V$_S$)

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{577}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{4.92 \times 1{,}000}{580}\right) \times 68 = 577\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76 + 0) = 57$ kips

$V_{tsu} = 101$ kips > $ØV_n = 57$ kips ∴ 58 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.875}{4} = 62$ Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"} = 4.25$

A_v = 3/8" stud = 0.11 in^2

$s_{max} = d/2 = 8.875/2 = 4.4$ in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{4.25 \times 0.11 \times 60 \times 8.875}{58} = 4.3$ in

∴ $s_{used} = 4$ in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76 + 62) = 104$ Kips

$ØV_n = 104$ Kips > $V_{Tsu} = 101$ Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ mid\text{-}height} = 54$ Kips < $ØV_c = 57$ The rails go up the entire wall of the Shear Wall

4th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$), $P_u = 3.93$ k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{461}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000 = 76$ kips

$N_u = \left(\frac{P_u \times 1{,}000}{I_w}\right) \times b = \left(\frac{3.93 \times 1{,}000}{580}\right) \times 68 = 461\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76+0) = 57$ kips

$V_{tsu} = 100$ kips > $ØV_n = 57$ kips ∴ 57 kips needed

Shear Capacity for New Shear Wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{461}{2{,}000\times68\times10}\right)\,1\sqrt{4000}\,\times68\times8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u\times1{,}000}{I_w})\times b = \left(\frac{3.93\times1{,}000}{560}\right)\times68 = 461\,lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76 + 0) = 57 kips

V_{tsu} = 100 kips > $ØV_n$ = 57 kips ∴ 57 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n\times A_v\times f_y\times d}{s} = \frac{4.25\times0.11\times60\times8.875}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"}$ = 4.25

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =8.875/2 = 4.4 in

$s_{needed} = \frac{n\times A_v\times f_y\times d}{V_S} = \frac{4.25\times0.11\times60\times8.875}{57}$ = 4.4 in

∴ s_{used} = 4 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76 + 62) = 104 Kips

$ØV_n$ = 104 Kips > V_{Tsu} = 100 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ Water\ Level}$= 17 Kips < $ØV_c$ = 57 The rails go up 18 in of the Shear Wall

5th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 2.95 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{346}{2{,}000\times68\times10}\right)\,1\sqrt{4000}\,\times68\times8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u\times1{,}000}{I_w})\times b = \left(\frac{2.95\times1{,}000}{580}\right)\times68 = 346\,lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76+0) = 57$ kips

$V_{tsu} = 2$ kips $< ØV_n = 57$ kips ∴ No shear studs needed

By inspection the remaining shear walls are adequate to resist the tsunami shear force.

A.15.3.5 Overall Wall loading:

Floor 1 Try 1 (Elevator):

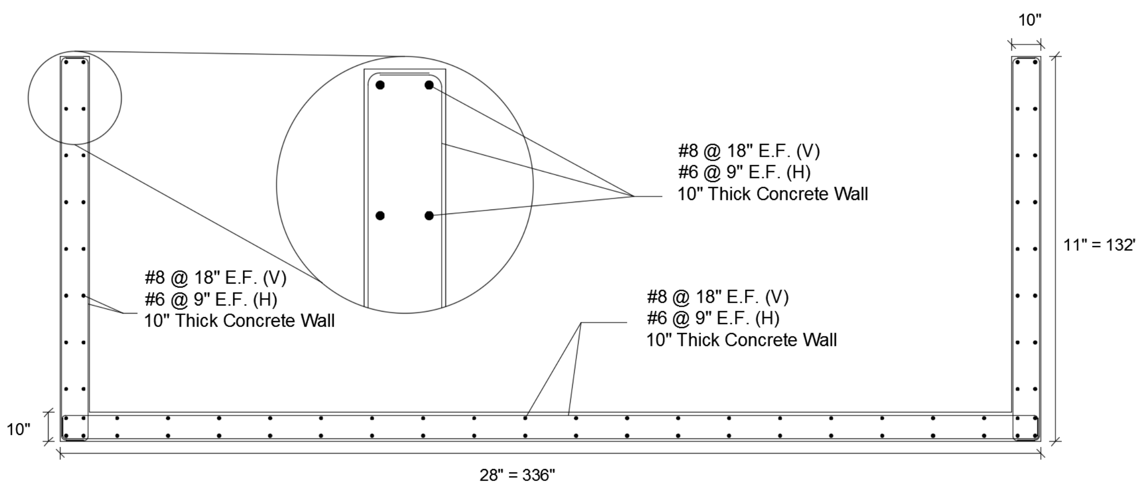

Figure A-107: Original Elevator/ Mech. Room shear wall cross-section at the ground floor level

Floor 2 Try 1 (Elevator):

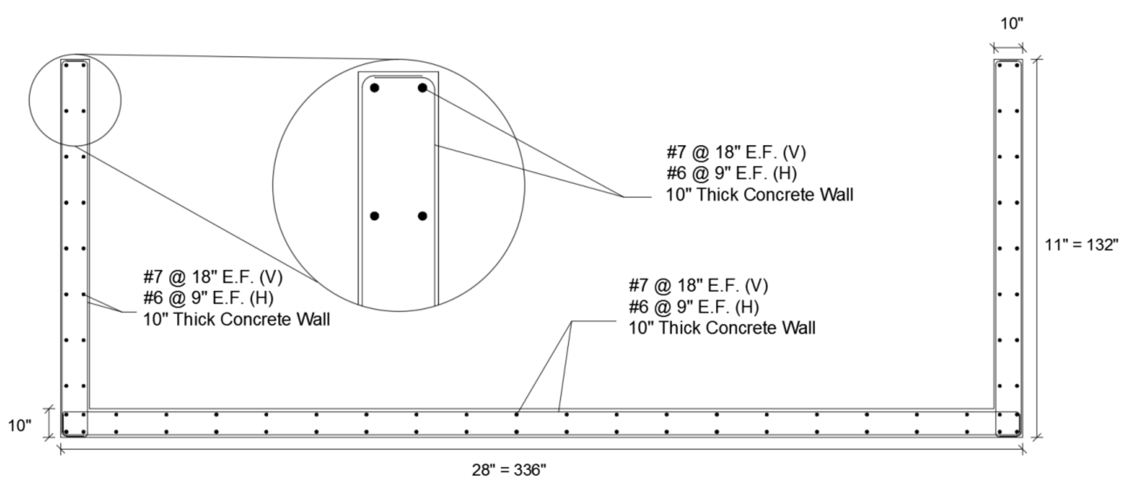

Figure A-108: Original Elevator/ Mech. Room shear wall cross-section at the 2nd floor level

Floor 3-7 Try 1 (Elevator):

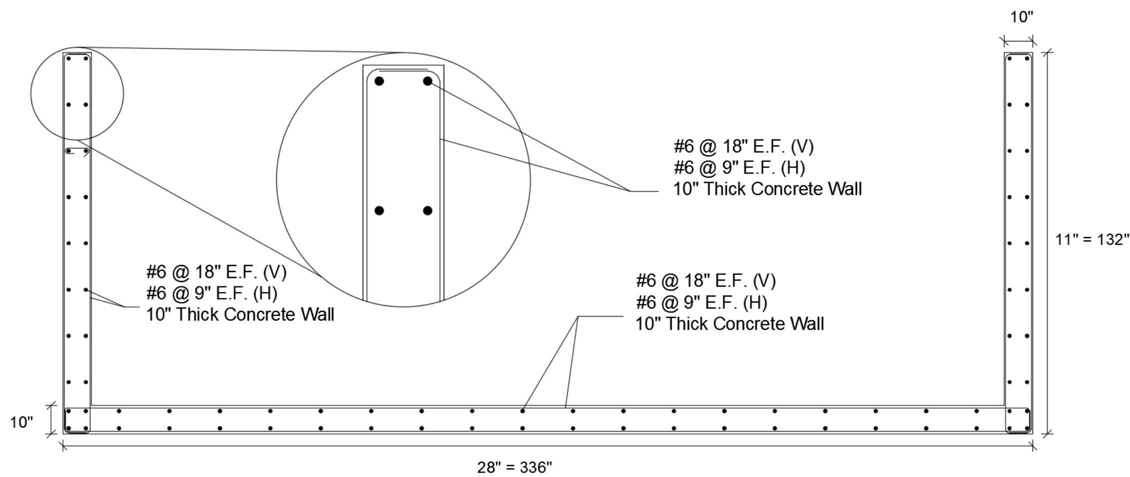

Figure A-109: Original Elevator/ Mech. Room shear wall cross-section at the 3rd floor level

Floor 1 Try 1 (Stairs):

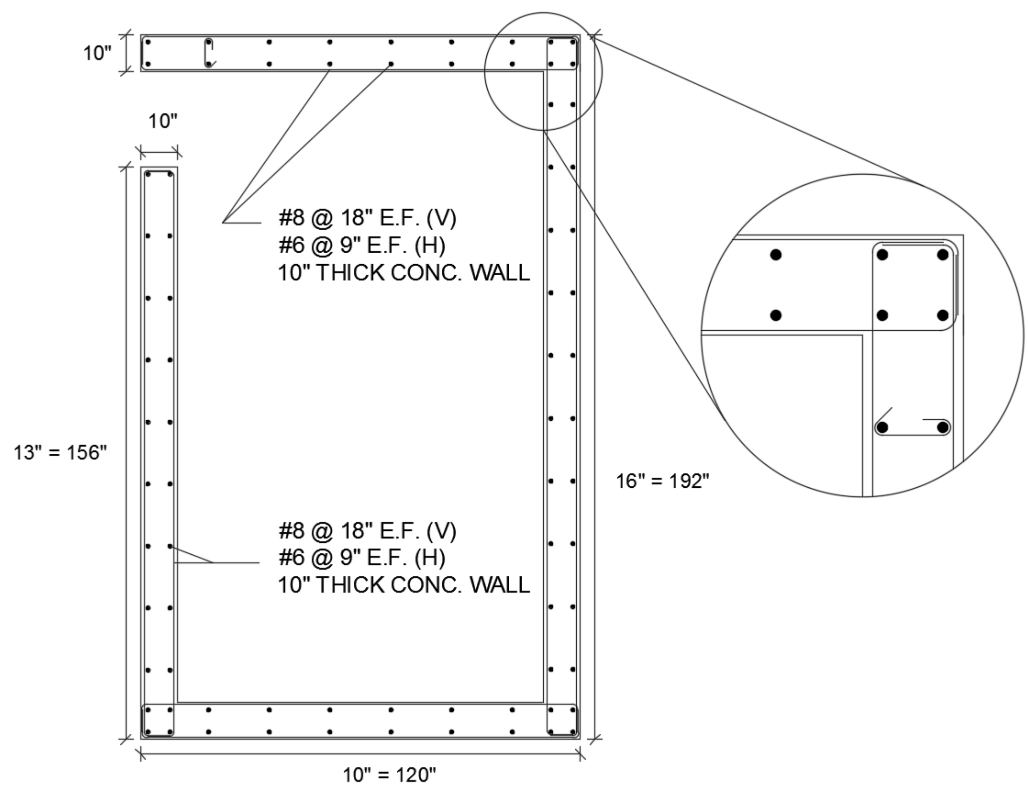

Figure A-110: Original stairwell shear wall cross-section at the ground floor level

Floor 2 Try 1 (Stairs):

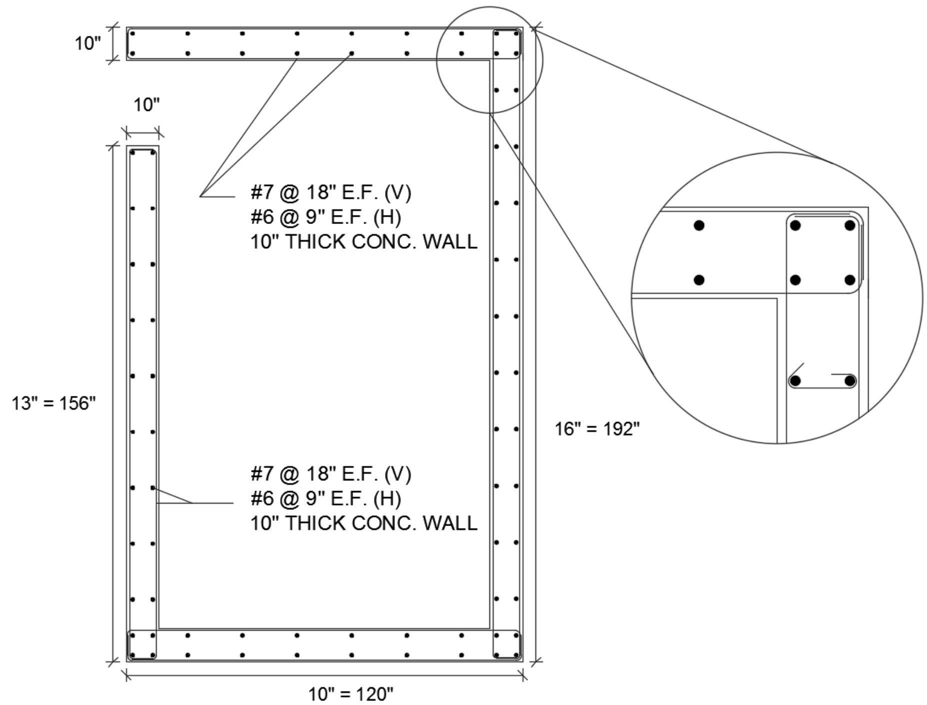

Figure A-111: Original stairwell shear wall cross-section at the 2nd floor level

Floor 3 Try 1 (Stairs):

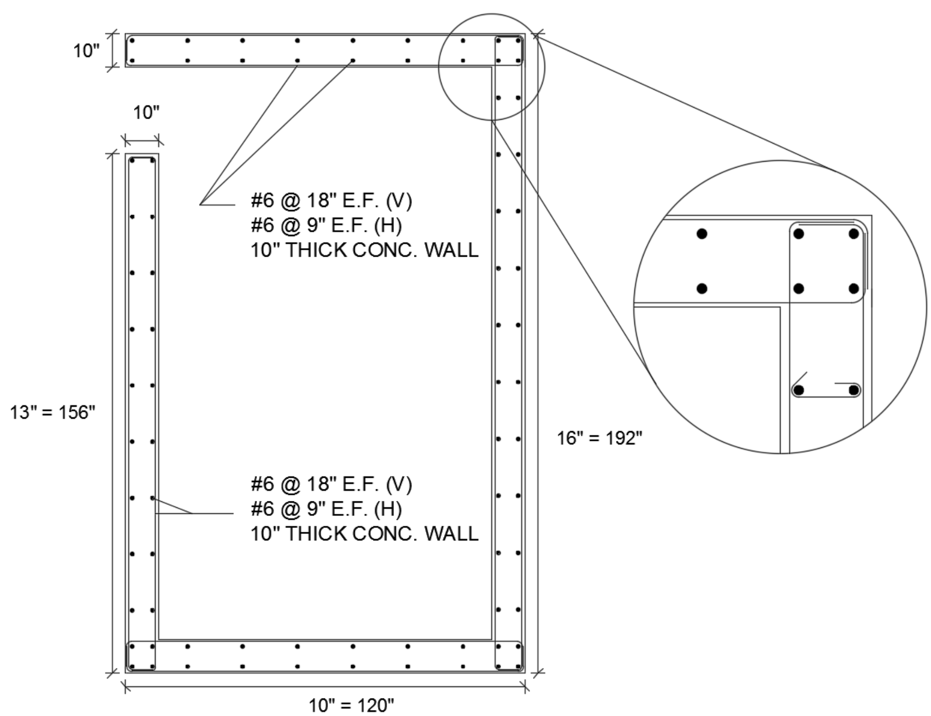

Figure A-112: Original stairwell shear wall cross-section at the 3rd floor level

A.15.3.6 Gravity Load Calculation (for completeness)

The gravity load tributary width is 5.5 ft. Gravity loads will be computed per foot width of wall.

The Dead Load at the base of the wall is:

$P_D = [128(5.5)(1)(6)+123(5.5)(1)+0.83(1)(150)(66)]/1000 = 13.1\ k/ft$

Floor Live load reduction: Reduction Factor $= 0.25+15/[1(5.5)(28)(6)]^{0.5} = 0.743$

$P_L = 0.743[55(5.5)(1)(6)]/1000 = 1.35\ k/ft$

Roof Live Load reduction: Reduction Factor $= R_1R_2= [1.2-(0.001)(5.5)(28)](1.0) = 1.05$*, Use 1.0*

$P_{Lr} = 20(5.5)(1)/1000 = 0.110\ k/ft$

Analysis of a 50 foot width of wall with the applied tsunami hydrodynamic loads from Load Case 2 results in the following bending moment and shear force diagrams:

Table A-9 summarizes the maximum critical load, bending moment and shear forces for all inundated shear walls using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and long impact (Impact). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table A-9: Results from loading conditions of Hilo residential building Overall shear walls (Floor 1 - 2)

Moment k-ft	Axial Load Kips	Shear Kips	Load Combination
Floor 1			
Earthquake			
5,753	630	161	1.2D+Ftsu+0.5L (Elv/Mech)
5,753	399	161	0.9D+Ftsu (Elv/Mech)
-5,753	630	161	1.2D+Ftsu+0.5L (Elv/Mech)
-5,753	399	161	0.9D+Ftsu (Elv/Mech)
16,390	796	340	1.2D+Ftsu+0.5L (Stairs)
16,390	566	340	0.9D+Ftsu (Stairs)
-16,390	796	340	1.2D+Ftsu+0.5L (Stairs)
-16,390	566	340	0.9D+Ftsu (Stairs)
Tsunami			
6,623	601	185	1.2D+Ftsu+0.5L (Elv/Mech)
6,623	371	185	0.9D+Ftsu (Elv/Mech)
-6,623	601	185	1.2D+Ftsu+0.5L (Elv/Mech)
-6,623	371	185	0.9D+Ftsu (Elv/Mech)
18,868	793	391	1.2D+Ftsu+0.5L (Stairs)
18,868	563	391	0.9D+Ftsu (Stairs)
-18,868	793	391	1.2D+Ftsu+0.5L (Stairs)
-18,868	563	391	0.9D+Ftsu (Stairs)
Floor 2			
Earthquake			
3,930	539	138	1.2D+Ftsu+0.5L (Elv/Mech)
3,930	342	138	0.9D+Ftsu (Elv/Mech)
-3,930	539	138	1.2D+Ftsu+0.5L (Elv/Mech)
-3,930	342	138	0.9D+Ftsu (Elv/Mech)
12,310	686	337	1.2D+Ftsu+0.5L (Stairs)
12,310	489	337	0.9D+Ftsu (Stairs)
-12,310	686	337	1.2D+Ftsu+0.5L (Stairs)
-12,310	489	337	0.9D+Ftsu (Stairs)
Tsunami			
4,525	515	159	1.2D+Ftsu+0.5L (Elv/Mech)
4,525	317	159	0.9D+Ftsu (Elv/Mech)
-4,525	515	159	1.2D+Ftsu+0.5L (Elv/Mech)
-4,525	317	159	0.9D+Ftsu (Elv/Mech)
14,171	684	388	1.2D+Ftsu+0.5L (Stairs)
14,171	486	388	0.9D+Ftsu (Stairs)
-14,171	684	388	1.2D+Ftsu+0.5L (Stairs)
-14,171	486	388	0.9D+Ftsu (Stairs)

A.15.3.7 Existing Exterior Shear Wall Design for Combined Gravity and Tsunami Loads

The shear wall chosen at Grid Line D and Grid Line 10 from **Figure A-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**.

Figure A-113 to **Figure A-116** shows the interaction diagram for the typical exterior shear wall including the tsunami load combinations.

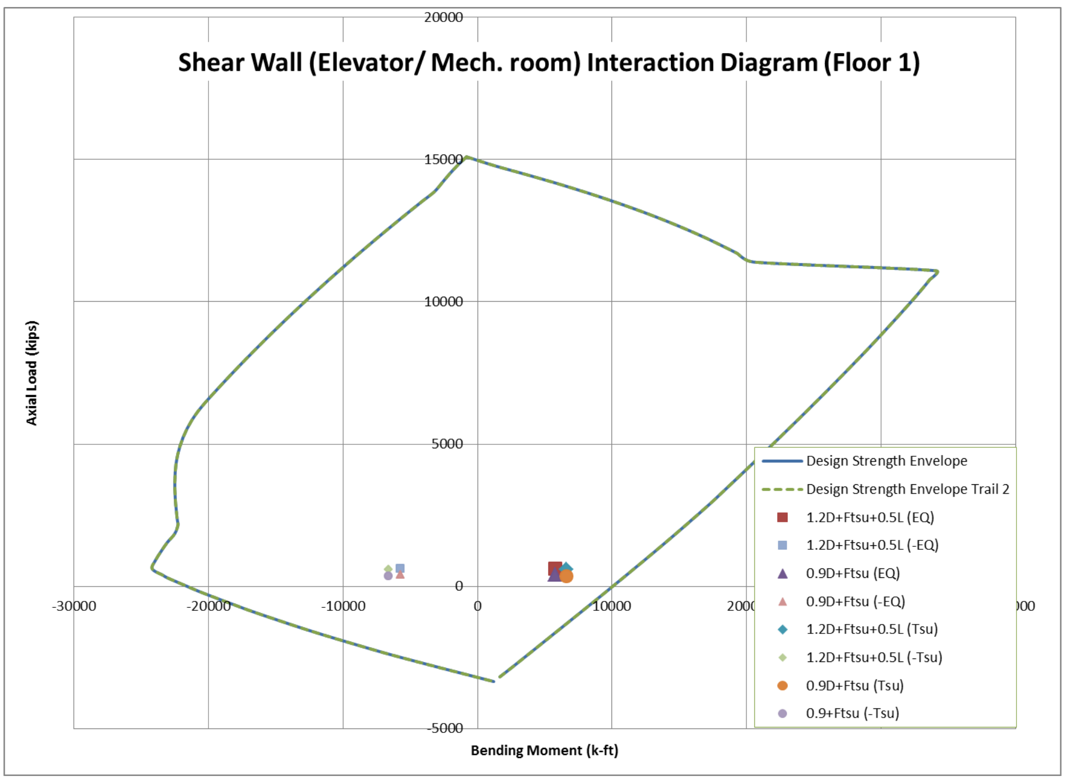

Figure A-113: Interaction diagram for typical ground floor overall elevator shear wall showing tsunami load combinations

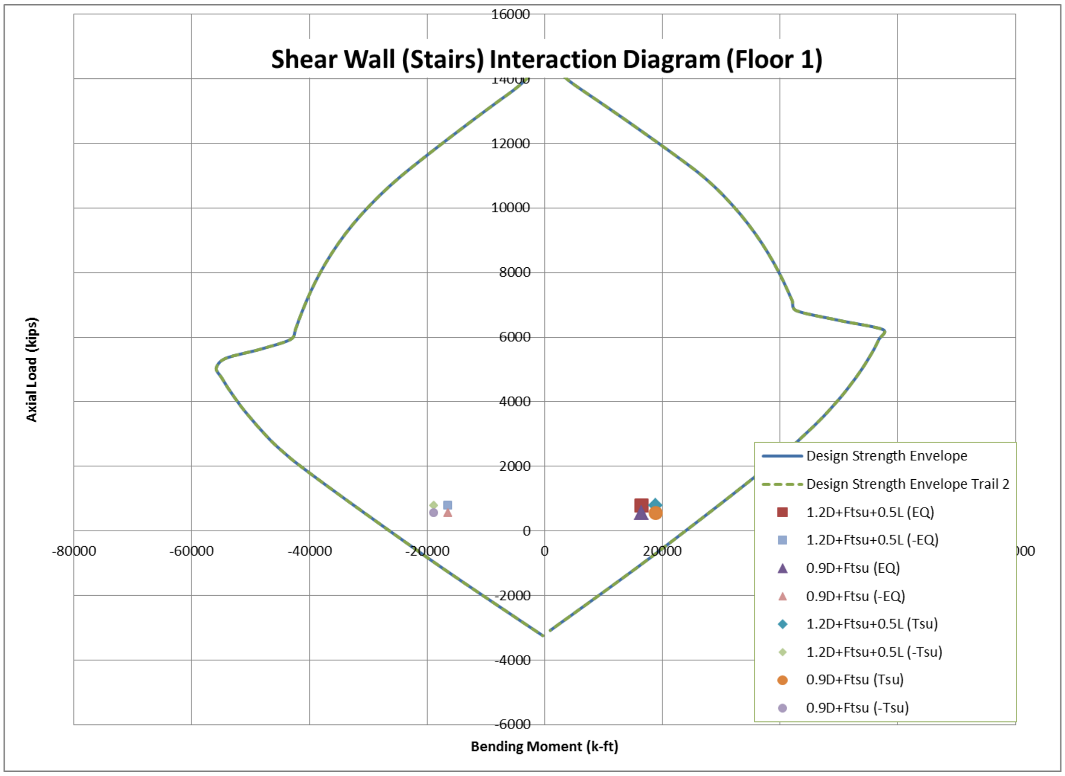

Figure A-114: Interaction diagram for typical ground floor overall stair shear wall showing tsunami load combinations

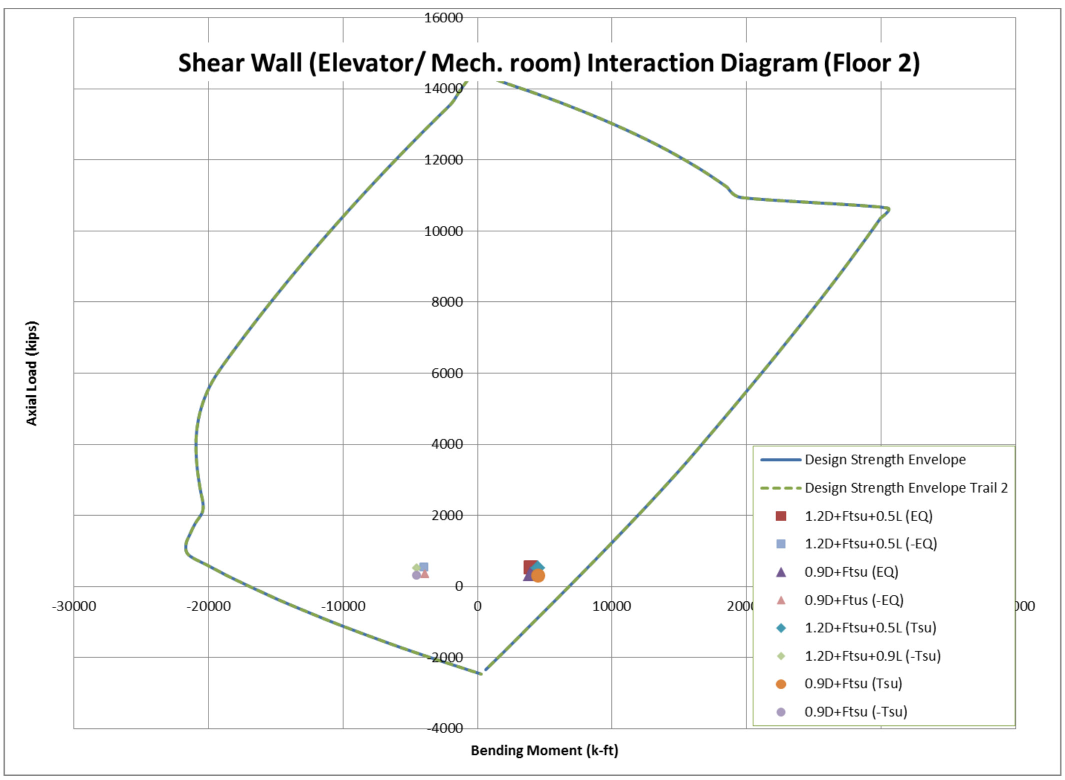

Figure A-115: Interaction diagram for typical 2nd floor overall elevator shear wall showing tsunami load combinations

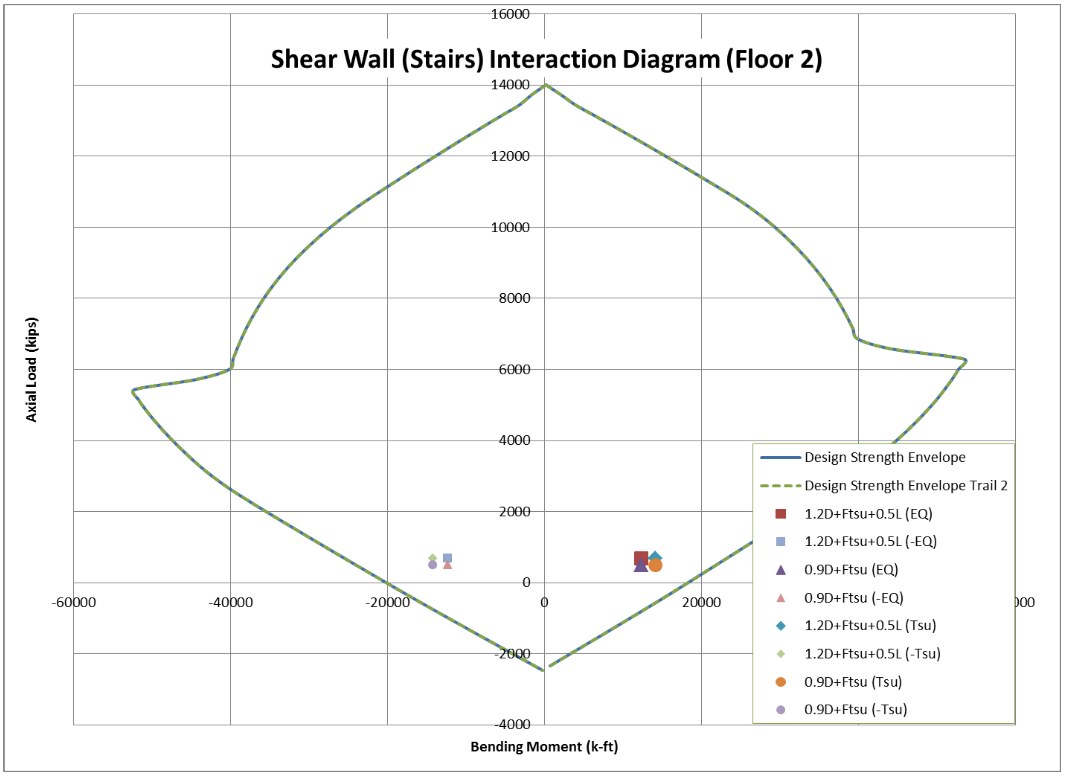

Figure A-116: Interaction diagram for typical 2nd floor overall stair shear wall showing tsunami load combinations

A.15.3.8 New Typical Shear Wall Design

The interaction diagrams show that the walls on floors 1 to 2 are inadequate for the bending moments due to hydrodynamic load on the overall shear walls. **Figure A-117** to Error! Reference source not found. show the revised wall designs required to resist the tsunami loads.

Floor 1 Try 2 (Elevator):

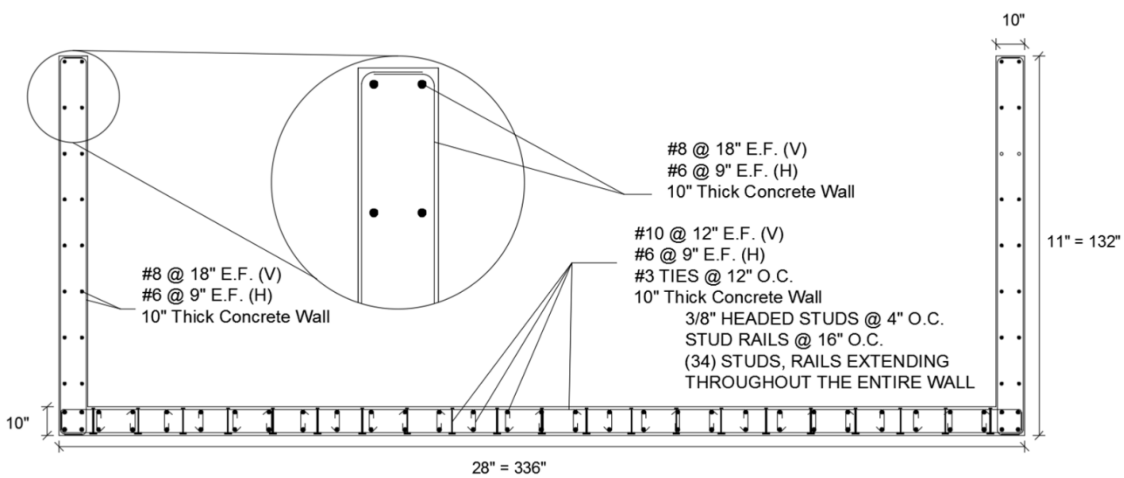

Figure A-117: New Elevator/ Mech. Room shear wall cross-section at the ground floor level

Floor 2 Try 2 (Elevator):

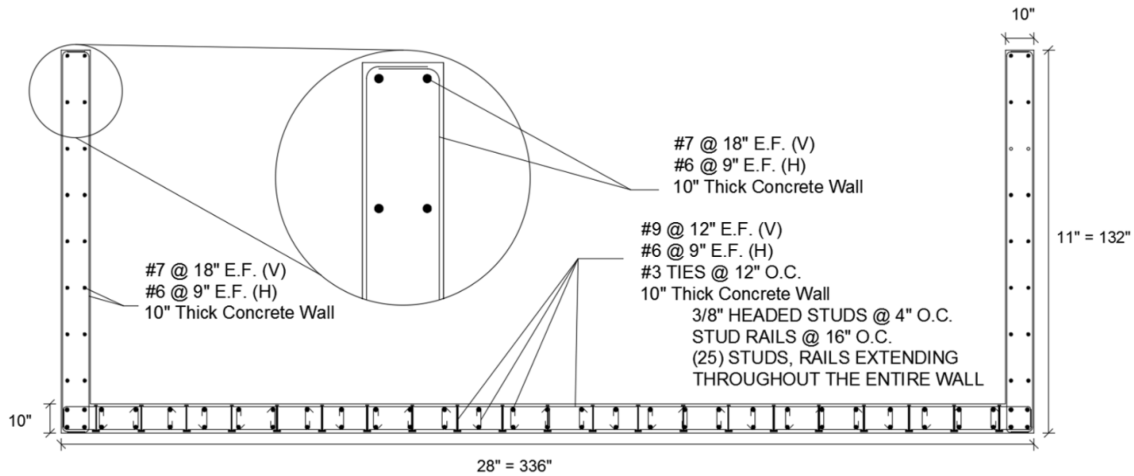

Figure A-118: New Elevator/ Mech. Room shear wall cross-section at the 2nd floor level

Floor 3 Try 2 (Elevator):

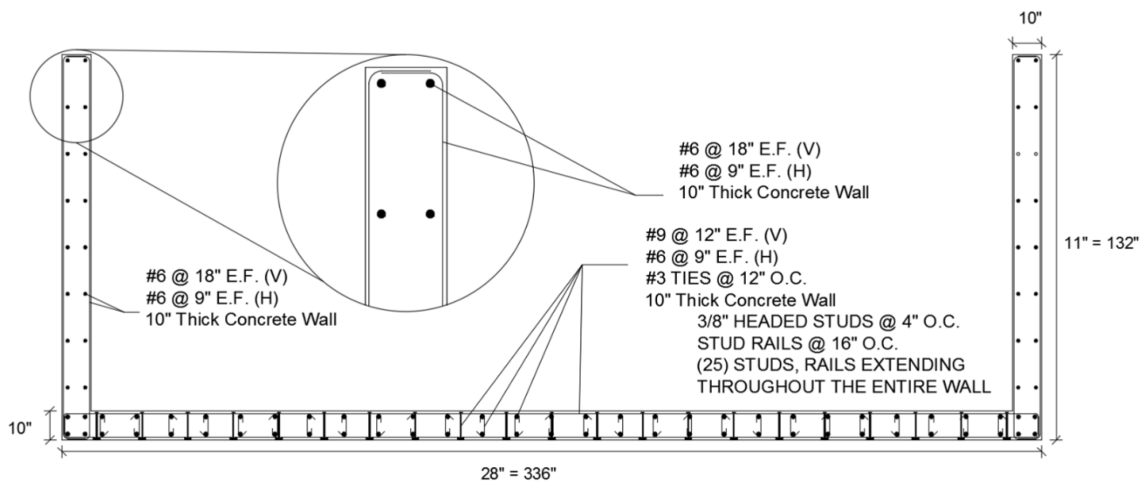

Figure A-119: New Elevator/ Mech. Room shear wall cross-section at the 3rd floor level

Floor 4 Try 2 (Elevator):

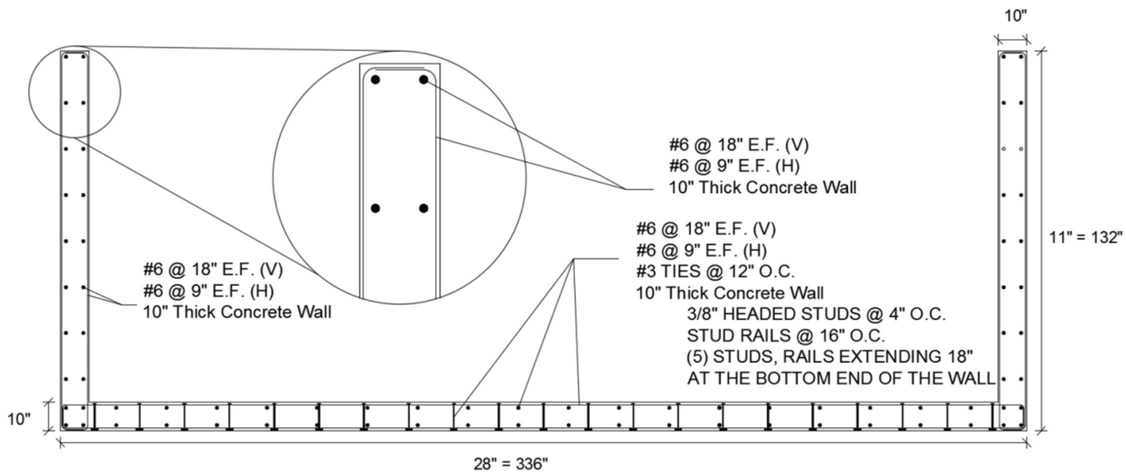

Figure A-120: New Elevator/ Mech. Room shear wall cross-section at the 4th floor level

A.15.3.9 Exterior Shear Wall Shear Design

Critical Shears:

1st Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$)

Shear Capacity of existing shear wall (Elevator):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\,\lambda \sqrt{f'_c}\, h\, d = 2 \times 1\, \sqrt{4000}\ \times 10 \times 105.6/1{,}000 = 267$ kips

$d = 0.8 \times L_w = 0.8 \times 132" = 105.6\ in$

L_w = 11'= 132 in

h = 10" (Thickness)

Ø = 0.75

V_{tsu} = 184 kips > $ØV_c$ = 200 kips ∴ -21 kips needed

$V_S = \frac{A_t f_y d}{s} = \frac{(2\times0.44)\times60{,}000\times105.6}{9}$ = 1,239 kips

A_t = 0.44 in (#6 Rebar)

S = 9 in (Spacing)

$ØV_n = Ø(V_C + V_S)$ = 0.75 (267+1,239) = 1,130 kips

At d: V_u = 184 k < ϕV_n = 1,130 k, therefore the wall is adequate for shear.

Shear Capacity of existing shear wall (Stairs):

$ØV_n = Ø(V_C + V_S)$

Where $V_{C1} = 2\,\lambda\,\sqrt{f'_c}\,h\,d = 2 \times 1\,\sqrt{4000}\,\times 10 \times 154/1{,}000$ = 194 kips

Where $V_{C2} = 2\,\lambda\,\sqrt{f'_c}\,h\,d = 2 \times 1\,\sqrt{4000}\,\times 10 \times 125/1{,}000$ = 158 kips

$V_C = V_{C1} + V_{C2}$ = 194 + 158 = 352 kips

$d_1 = 0.8 \times L_w = 0.8 \times 192" = 154\ in$

$d_2 = 0.8 \times L_w = 0.8 \times 156" = 125\ in$

L_{w1} = 16'= 192 in

L_{w2} = 13'= 156 in

h = 10" (Thickness)

Ø = 0.75

V_{tsu} = 391 kips > $ØV_c$ = 264 kips $\therefore$ 170 kips needed

$V_{S1} = \frac{A_t f_y d}{s} = \frac{(2\times0.44)\times60{,}000\times154}{9}$ = 901 kips

$V_{S2} = \frac{A_t f_y d}{s} = \frac{(2\times0.44)\times60{,}000\times125}{9}$ = 732 kips

$V_s = V_{s1} + V_{s2}$ = 1,633 + 1,225 = 1,633 kips

A_t = 0.44 in (#6 Rebar)

S = 9 in (Spacing)

$ØV_n = Ø(V_C + V_S)$ = 0.75 (352+1,633) = 1,489 kips

At d: V_u =391k < ϕV_n = 1,489 k, therefore the wall is adequate for shear.

By inspection the remaining floors are adequate to resist the tsunami shear force.

B. Monterey Design Example

B.1 Project Site

The Monterey design example considers a multi-story reinforced concrete building in Monterey, California, at the location shown in **Figure B-1**. The center of the building footprint is located at 36.6000 N; 121.8821 W, which is 468 feet from the shoreline. **Figure B-1** also shows the three topographic transects along which the Energy Grade Line Analysis needs to be applied. The center transect, C, is drawn perpendicular to the shoreline, represented by the average coastline for 500 feet either side of the center transect. The clockwise, CW, and counterclockwise, CCW, transects are generated by rotating the center transect through 22.5 degrees in each direction, about the geometric center of the building plan at the grade plane (ASCE 7 Section 6.8.6.1). Each transect is then extended till it reaches the runup points on the ASCE 7 Tsunami Design Zone map. If the end of a transect falls between two of the runup points, then the runup elevations can be interpolated. The resulting runup elevations for each transect are given in **Table B-1** along with the approximate inundation limit distances obtained by Google Earth. These inundation limit distances will be revised once the runup elevations are plotted on the respective topographic profiles.

Note that for the States of Washington, Oregon and California, the ASCE 7 TDZ maps provide the runup elevations in relation to Mean High Water, MHW, and NAVD88. At the Monterey location the difference between these two elevation models is 4.77 feet. This difference varies along the Pacific Coast.

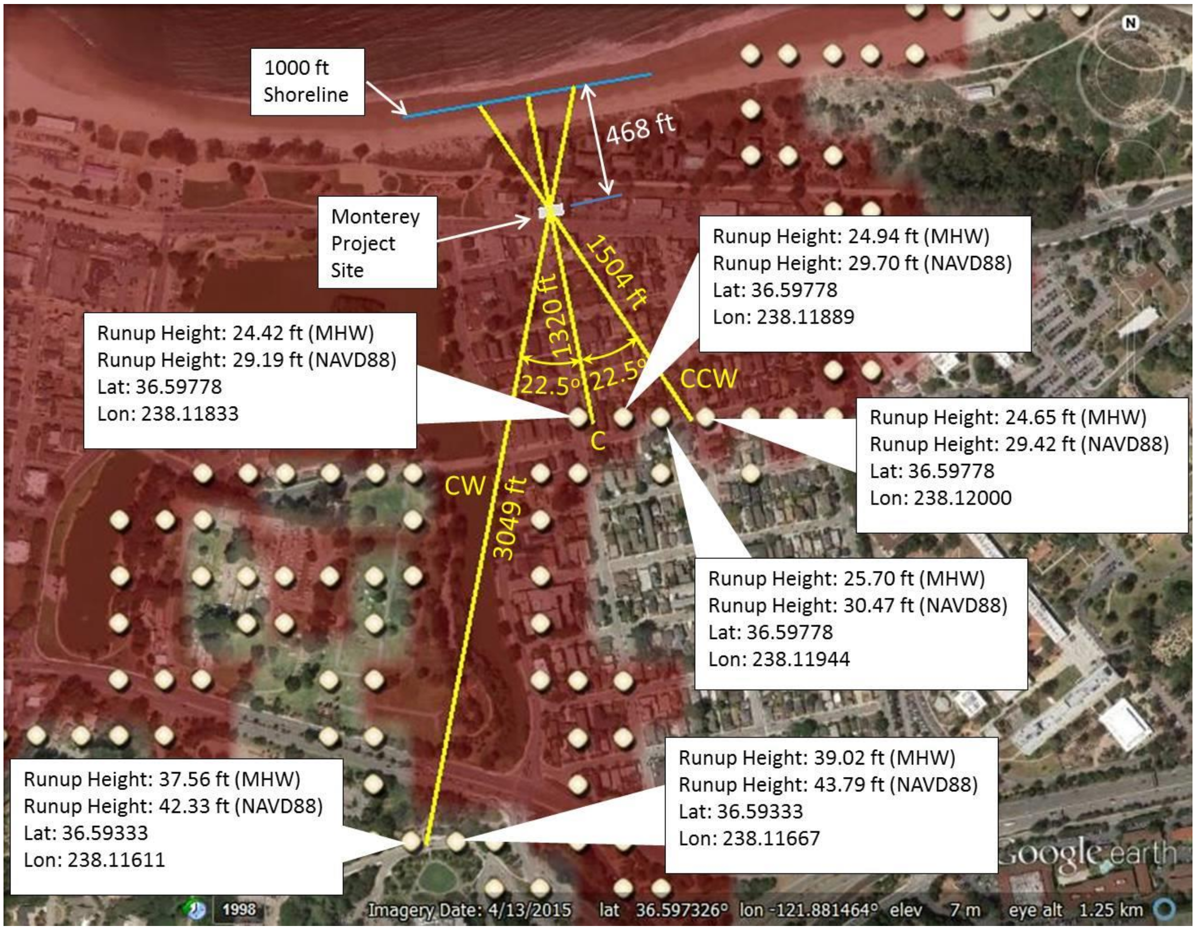

Figure B-1: Location of project site in Monterey, California, relative to inundation line defined by ASCE7-16 Tsunami Design Zone Map. The 22.5° variation in principal flow direction required by Section 6.8.6.1 results in Clockwise (CW) and Counterclockwise (CCW) transects on either side of the Center (C) transect.

Table B-1: Runup elevation and inundation limits for three transects through the Monterey Project site.

Transect	Runup Elevation (ft)				Inundation Limit (ft)	
	MHW Reference		NAVD88 Reference		From Google Earth	From WGS 84 Transect
	From TDZ	Incl. Sea Level Rise	From TDZ	Incl. Sea Level Rise		
Center	24.59	25.03	29.36	29.80	1320	1489
Counterclockwise	25.00	25.44	29.77	30.11	1504	1652
Clockwise	38.05	38.49	42.82	43.26	3049	3101

B.2 Sea Level Change – Section 6.5.3

ASCE 7 Section 6.5.3 requires that any anticipated sea level rise be included in the runup elevation used in the tsunami design. For this example, we will assume sea level change based on a 50 year project life cycle. ASCE 7 Commentary Section C6.5.3 provides a link to http://tidesandcurrents.noaa.gov/sltrends for historical sea level trends relative to mean sea level (MSL).

From the referenced website the following information is obtained:

"*Monterey, California; 9413450*

The mean sea level trend is 1.34 mm/year with a 95% confidence interval of +/- 1.35 mm/year based on monthly mean sea level data from 1973 to 2006."

The tsunami design should therefore consider the extrapolated prediction of 2.69 mm/year over the 50 year project life cycle. This results in a sea level rise of 134.5 mm or 5.3" (0.44 ft). This must be added to the runup elevation for use in the Energy Grade Line Analysis, as shown in **Table B-1**.

B.3 Topographic Profiles

The topographic profiles along each of these transects was obtained from a Digital Elevation Model, DEM, with the following datums and resolution:

Horizontal Datum: WGS 84

Vertical Datum: MHW

Resolution: 1/3 sec (approximately 10)

The topographic profiles are shown for the Center, Counterclockwise and Clockwise transects in **Figure B-2, Figure B-3**, and **Figure B-4** respectively. A horizontal line is plotted on each profile representing the runup elevation (including sea level rise) for each of these transects relative to the MHW datum from Table B-1. The point where this line intersects the profile represents the inundation limit and the starting point for the Energy Grade Line Analysis. The resulting inundation limit should be cross-checked with the Tsunami Design Zone map inundation line to ensure that they are similar distances from the shoreline (See **Table B-1**). If the TDZ inundation is significantly greater than the first intersection of the runup elevation line with the topographic profile, it may indicate that a region of high ground is present in the inundation zone. The runup elevation must then be modified to match this high ground elevation and the corresponding inundation limit determined where the modified runup elevation next intersects the topographic profile. The resulting values for inundation limit are shown in **Table B-1** and are used in the EGLA along each transect.

The project site location is also indicated on each plot. For the center transect, the site is located 468 feet from the shoreline (**Figure B-2**). The elevations at the project site vary slightly for the three transects, which can be attributed to slight differences in the elevation data points used to generate each transect profile.

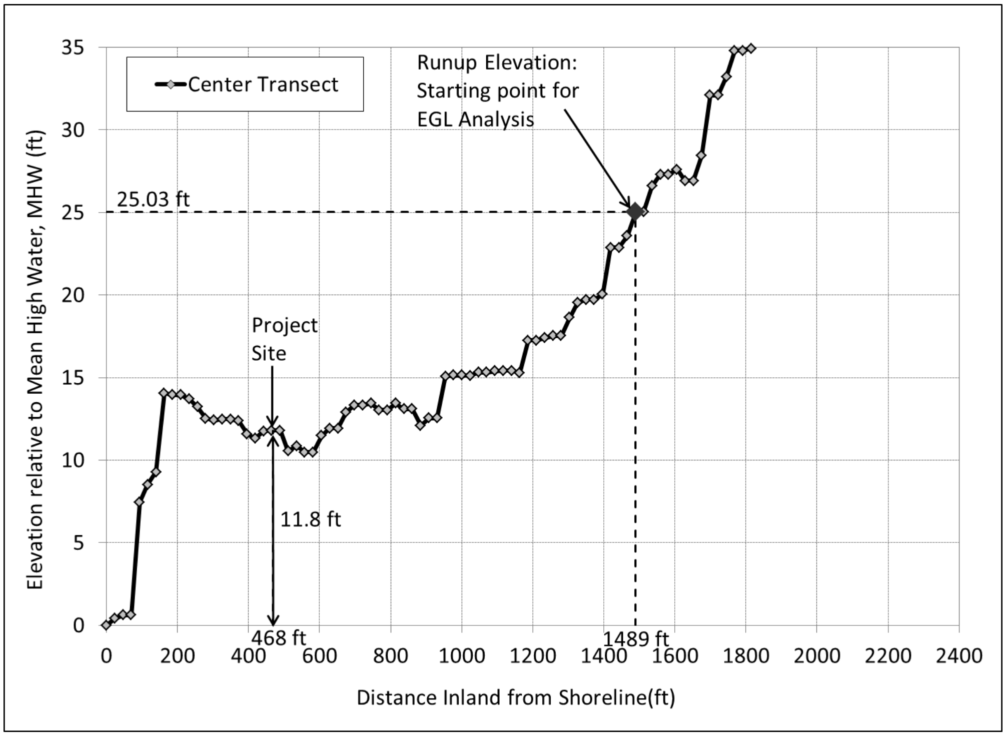

Figure B-2: Topographic profile for Center transect

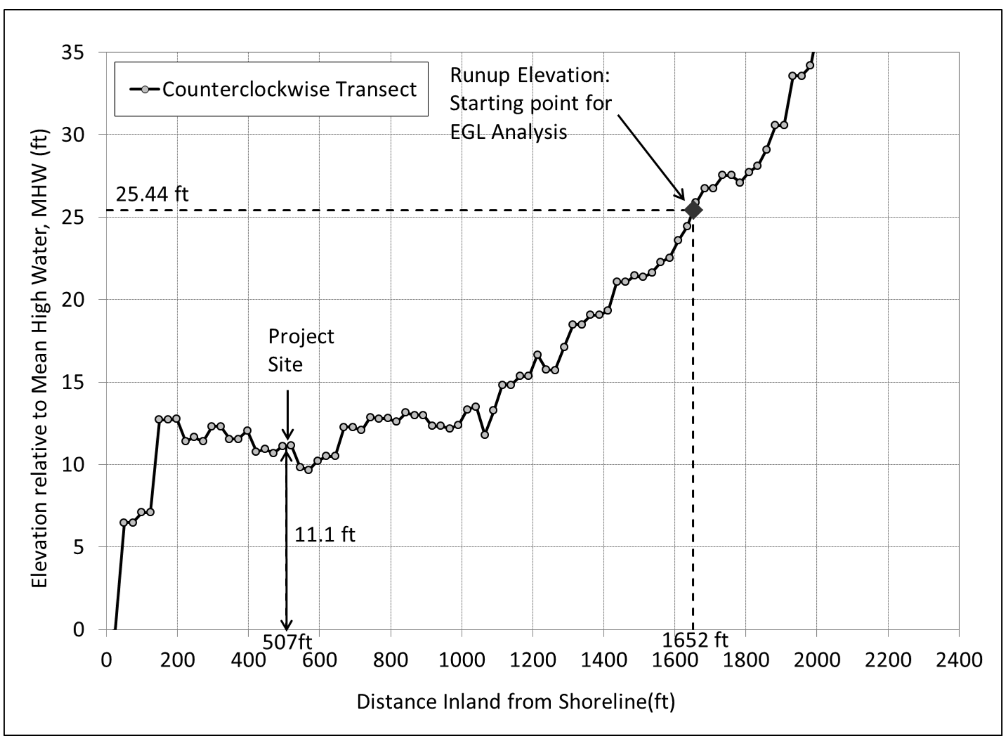

Figure B-3: Topographic profile for Counterclockwise transect

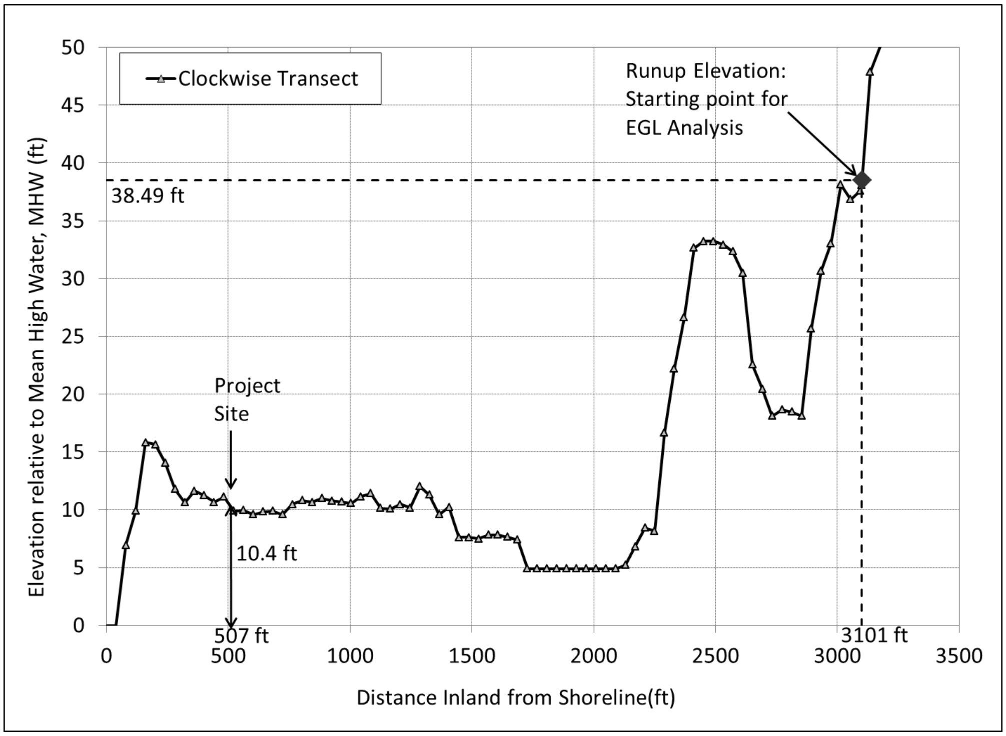

Figure B-4: Topographic profile for Clockwise transect

B.4 Tsunami Bore Determination

In order to determine whether or not a tsunami bore must be considered at the project site, the conditions in ASCE 7 Section 6.6.4 are evaluated for each transect. Tsunami bores shall be considered where any of the following conditions exist:

8. Prevailing nearshore bathymetric slope is 1/100 or milder – YES (See **Figure B-5** and associated discussion).
9. Shallow fringing reefs or other similar step discontinuities – Does not apply.
10. Where historically documented – Does not apply.
11. As described in the Recognized Literature – Does not apply.
12. As determined by a site-specific inundation analysis – not required for TRC II buildings.
    Therefore bore loading must be considered in this design.

Figure B-5 shows the approach to determining the average nearshore bathymetric slope so as to determine whether or not tsunami bores need to be considered per ASCE 7 Section 6.4.4. A central line is drawn perpendicular to the shoreline. This line is an extension of the center transect running through the project site. The distance from the shoreline to the 100 meter bathymetric line, indicated by the offshore data points in the ASCE offshore wave maps, is then used to determine the average nearshore bathymetric slope. If any of the transect lines does not intersect the 100 meter bathymetric line, this transect can be ignored for the purpose of determining whether or not there is a bore.

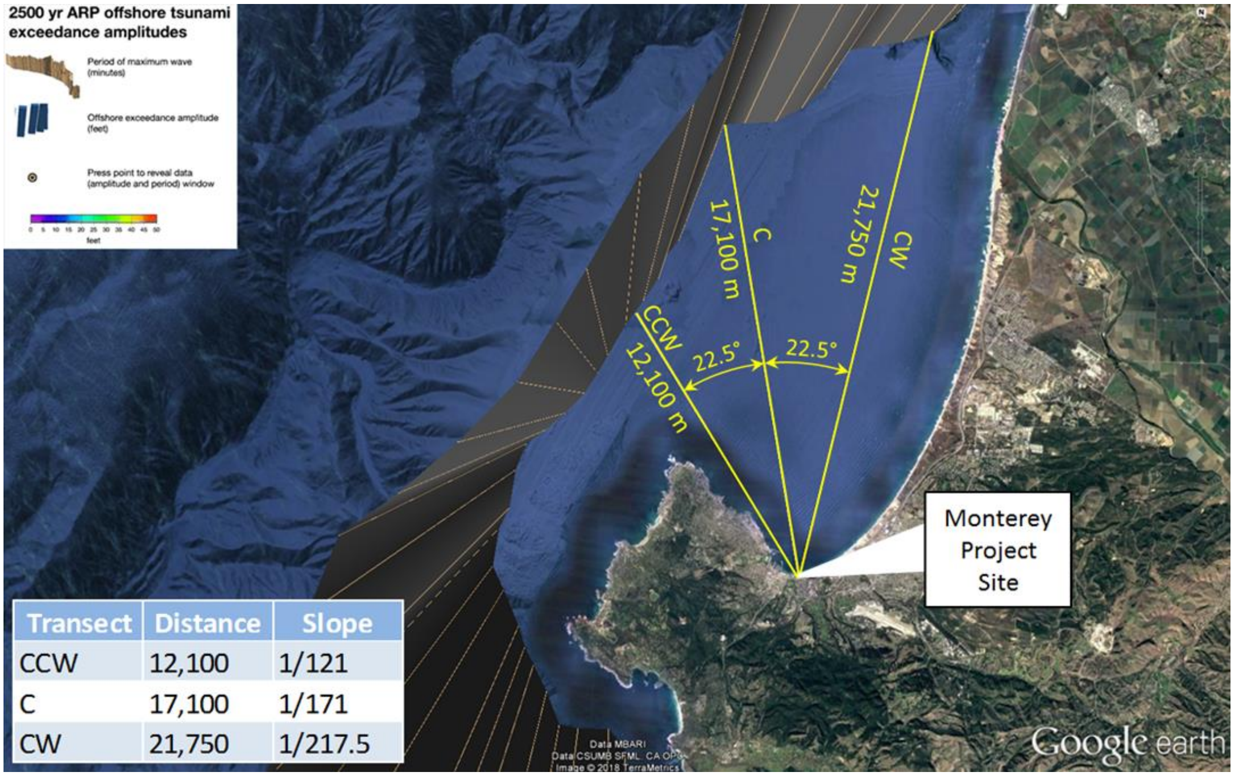

Figure B-5: Determination of average nearshore slope from 100 meter bathymetric line to shoreline along a line perpendicular to the shoreline and lines rotated 22.5 degrees to either side of the center line.

The average nearshore bathymetric slope is then computed using:

$$\emptyset = \frac{100}{distance} \quad \text{in meters}$$

or

$$\emptyset = \frac{328}{distance} \quad \text{in feet .}$$

The table in **Figure B-5** shows that all three offshore transect slopes are milder than 1/100, therefore this project site must consider the effect of tsunami bores along each transect. If only one or two of the transect slopes is milder than 1/100, then the site must consider tsunami bores along those transects because of the potential for a tsunami bore approaching from that direction.

B.5 Determination of Inundation Depth and Flow Velocity using EGLA

The Energy Grade Line Analysis (EGLA) is a stepwise procedure starting from the run up elevation at the mapped inundation limit, and working shoreward to get the flow parameters at the site of interest.

A spreadsheet was used to perform this operation along all three transects. The input values were the runup, including, referenced to MHW datum (**Table B-1** column 3), the inundation limit distance determined from the topographic profile (**Table B-1** column 5), a Manning's coefficient of 0.030 representing "all other cases" from ASCE 7 Table 6.6-1, and α = 1.3 representing bore conditions at the shoreline as specified in ASCE 7 Section 6.6.4. The resulting inundation depth profiles, both with and without the topographical elevation, are shown in **Figure B-6** and **Figure B-7** for the Center transect,

Figure B-8 and **Figure B-9** for the Counterclockwise transect, and **Figure B-10** and **Figure B-11** for the Clockwise transect.

Because of the similar transect lengths and runup elevations for the Center and Counterclockwise transects, the flow depth profiles are similar. The maximum flow depth at the project site is 10.8 or 11.7 feet based on these transects. However, for the longer Clockwise transect with larger runup elevation, the flow depth at the project site is 21.6 feet, which is the value of h_{max} that will be used in the subsequent design calculations.

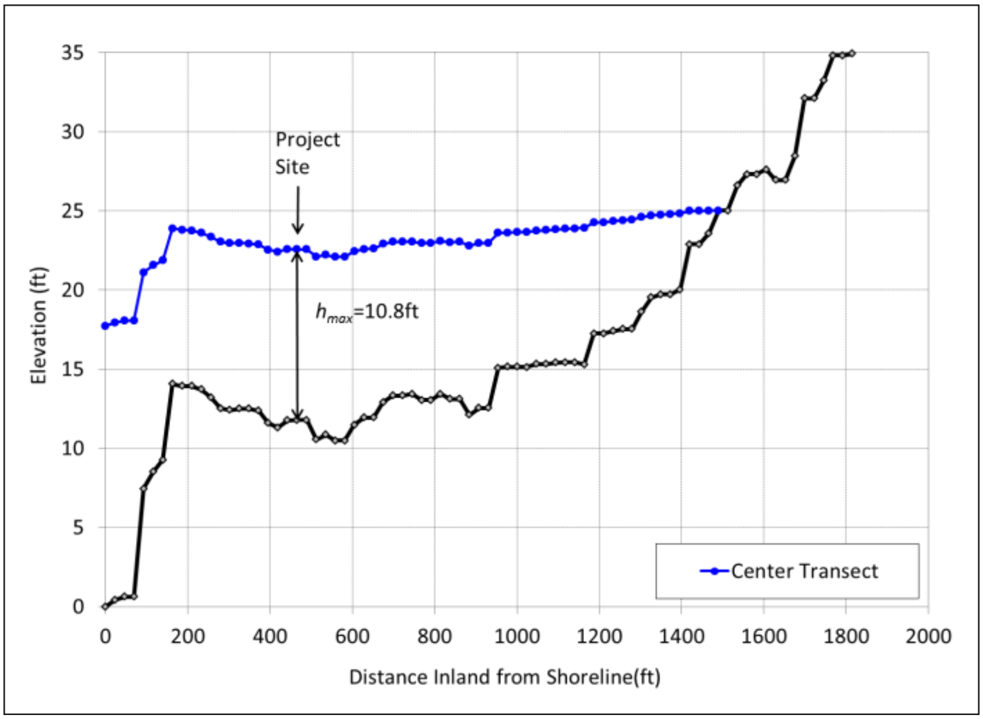

Figure B-6: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for center transect

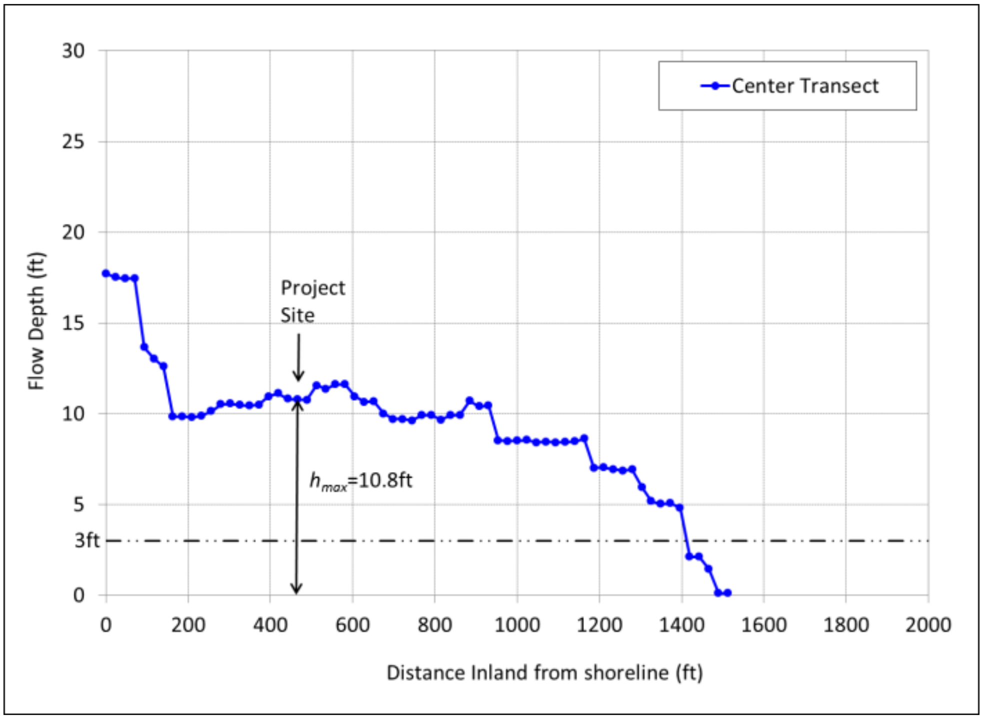

Figure B-7: Inundation depth (h_i) profile from Energy Grade Line analysis for center transect

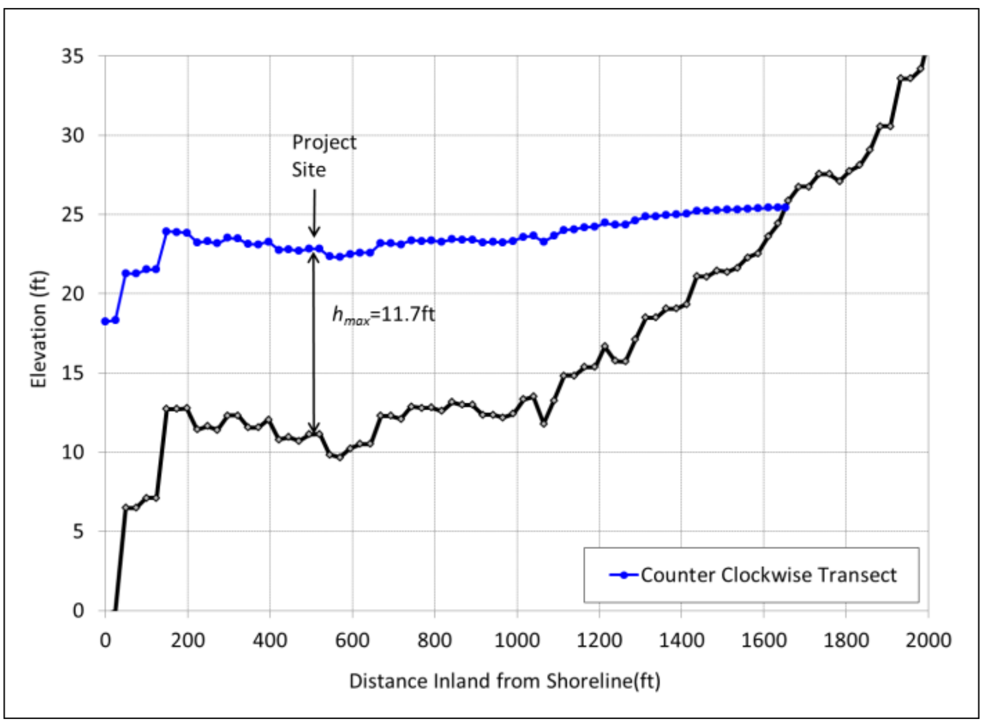

Figure B-8: Inundation depth (hi) over topographic transect from Energy Grade Line analysis for counterclockwise transect

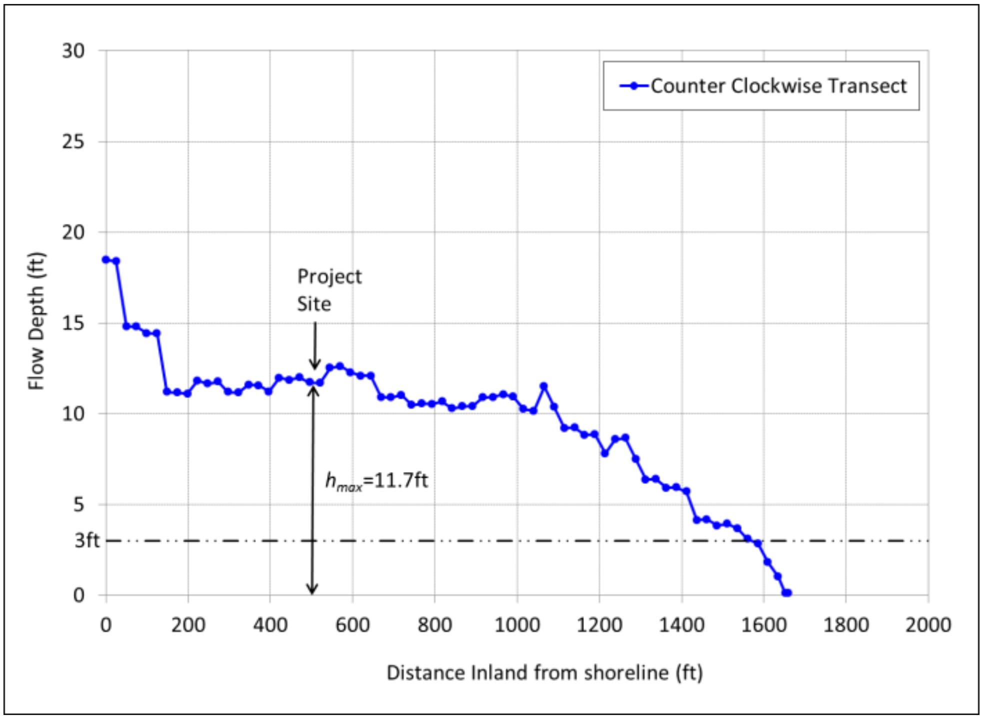

Figure B-9: Inundation depth (h_i) profile from Energy Grade Line analysis for counterclockwise transect

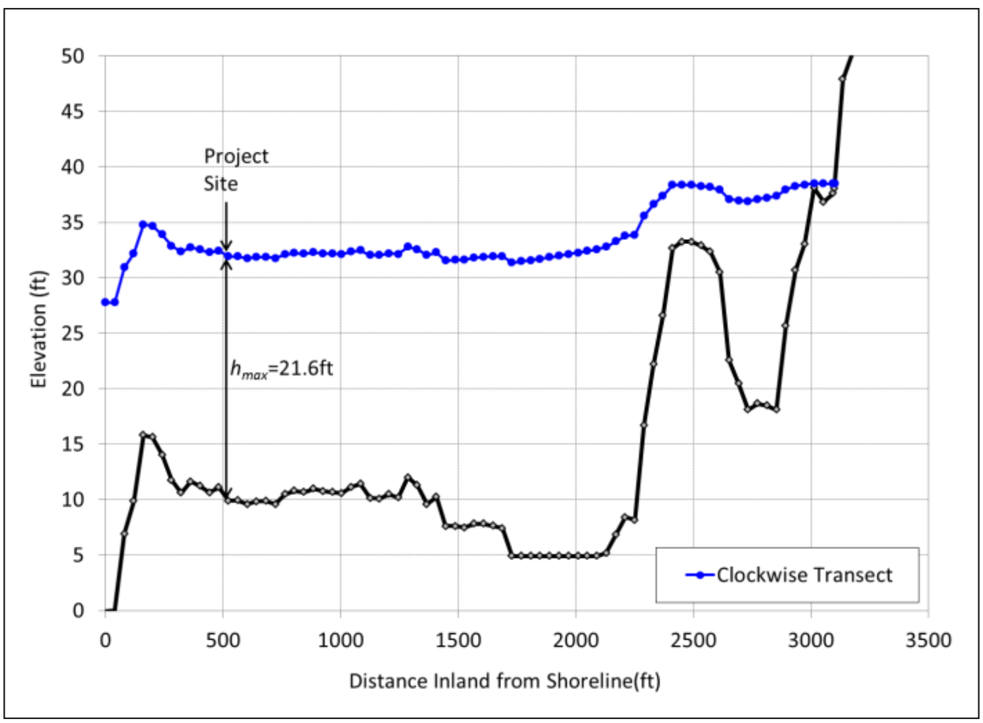

Figure B-10: Inundation depth (hi) over topographic transect from Energy Grade Line analysis for clockwise transect

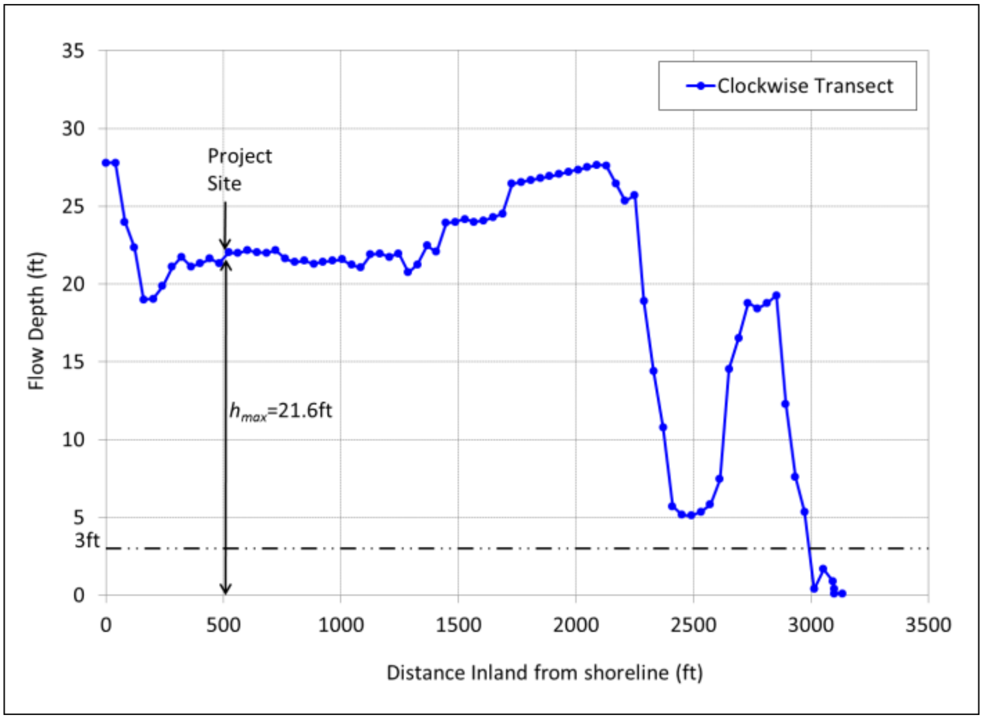

Figure B-11: Inundation depth (h_i) profile from Energy Grade Line analysis for clockwise transect

The flow velocity profiles across each transect as determined from the EGLA are shown in **Figure B-12**, **Figure B-13**, and **Figure B-14** for the Center, Counterclockwise and Clockwise transects, respectively. The minimum flow velocity that may be considered is 10 ft/sec, which is indicated on each of the plots. As with the flow depth, the Clockwise transect produces the largest estimate of flow velocity at 31.5 ft/sec, which is the value of u_{max} that will be used in the design calculations.

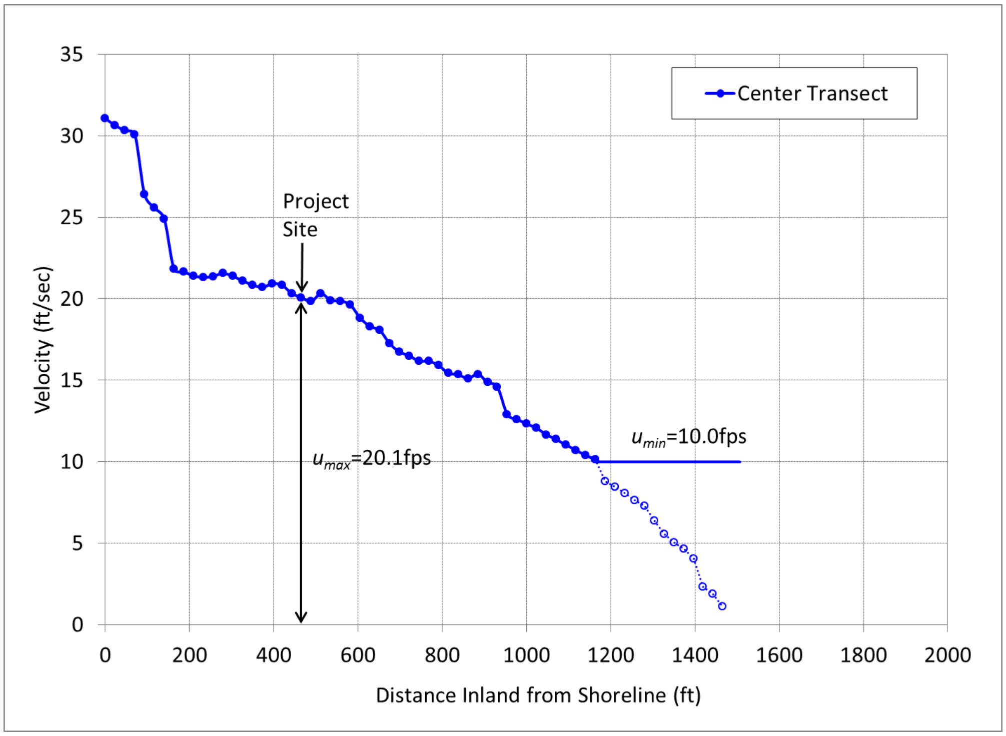

Figure B-12: Flow velocity (u_i) profile from Energy Grade Line analysis for Center transect

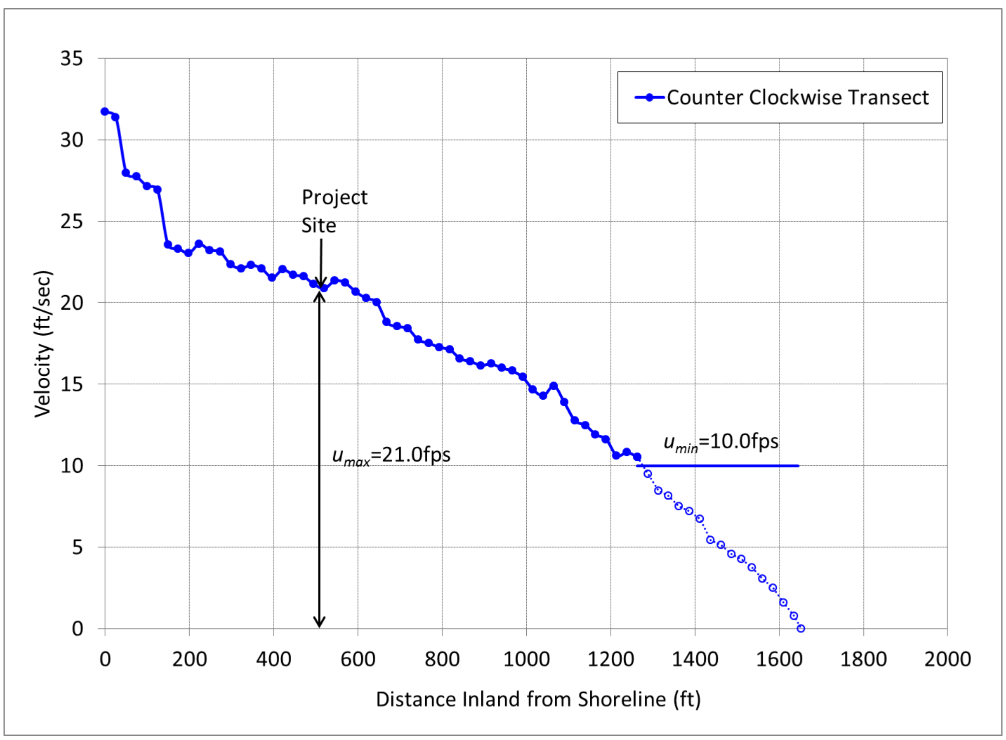

Figure B-13: Flow velocity (u_i) profile from Energy Grade Line analysis for Counterclockwise transect

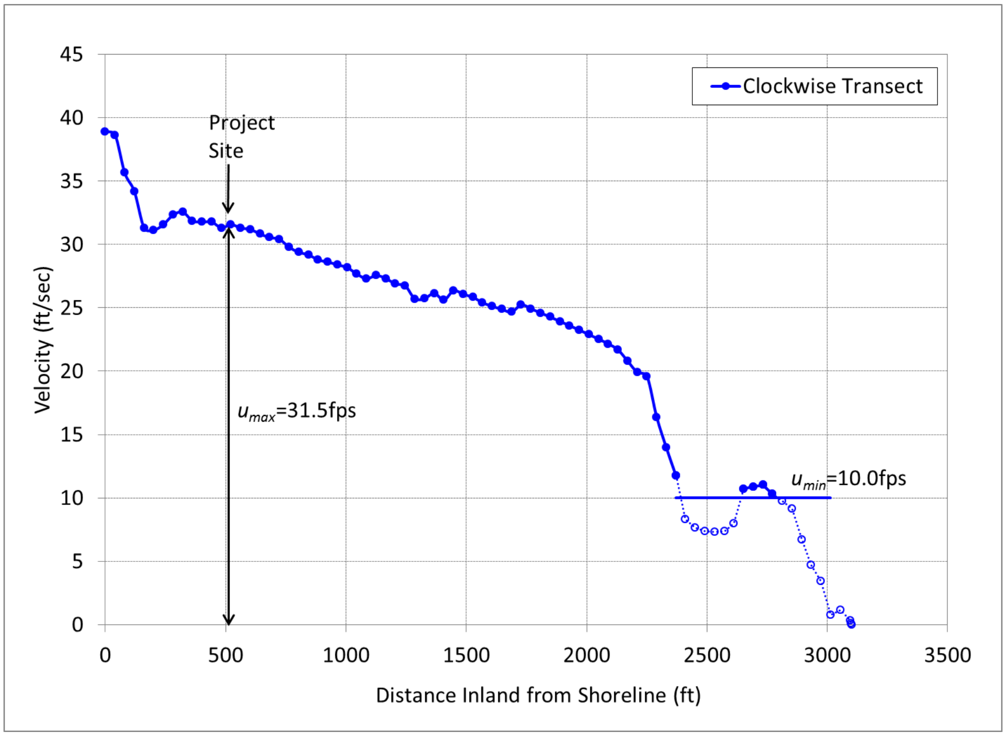

Figure B-14: Flow velocity (u_i) profile from Energy Grade Line analysis for Clockwise transect

All of the flow depths and flow velocities determined from the EGLA are listed in **Table B-2**

Table B-2: Results of Energy Grade Line Analysis for three transects through Monterey project site.

Transect	Maximum Flow Depth, h_{max} (ft)	Maximum Flow Velocity, u_{max} (ft/sec)
Center	10.8	20.1
Counterclockwise	11.7	21.0
Clockwise	**21.6**	**31.5**

B.6 Prototype Concrete Buildings

B.6.1 6-Story Office Building

The 6-story office building consists of a Special Moment Resisting Frame on the perimeter and selected interior frames, and interior gravity columns supporting posttensioned floor slabs (See **Figure B-15**). The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: D

Response Spectrum Parameters: S_s = 1.513, S_1 = 0.554, S_{DS} = 1.009, S_{D1} = 0.554

Structural System Response Factors: R = 8, Ω_o = 3, C_d = 5.5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 74 ft. With a maximum flow depth of 21.6 ft, this building could function as a "Refuge of Last Resort" at the 3rd level (26 ft) up to the roof (if acceptable).

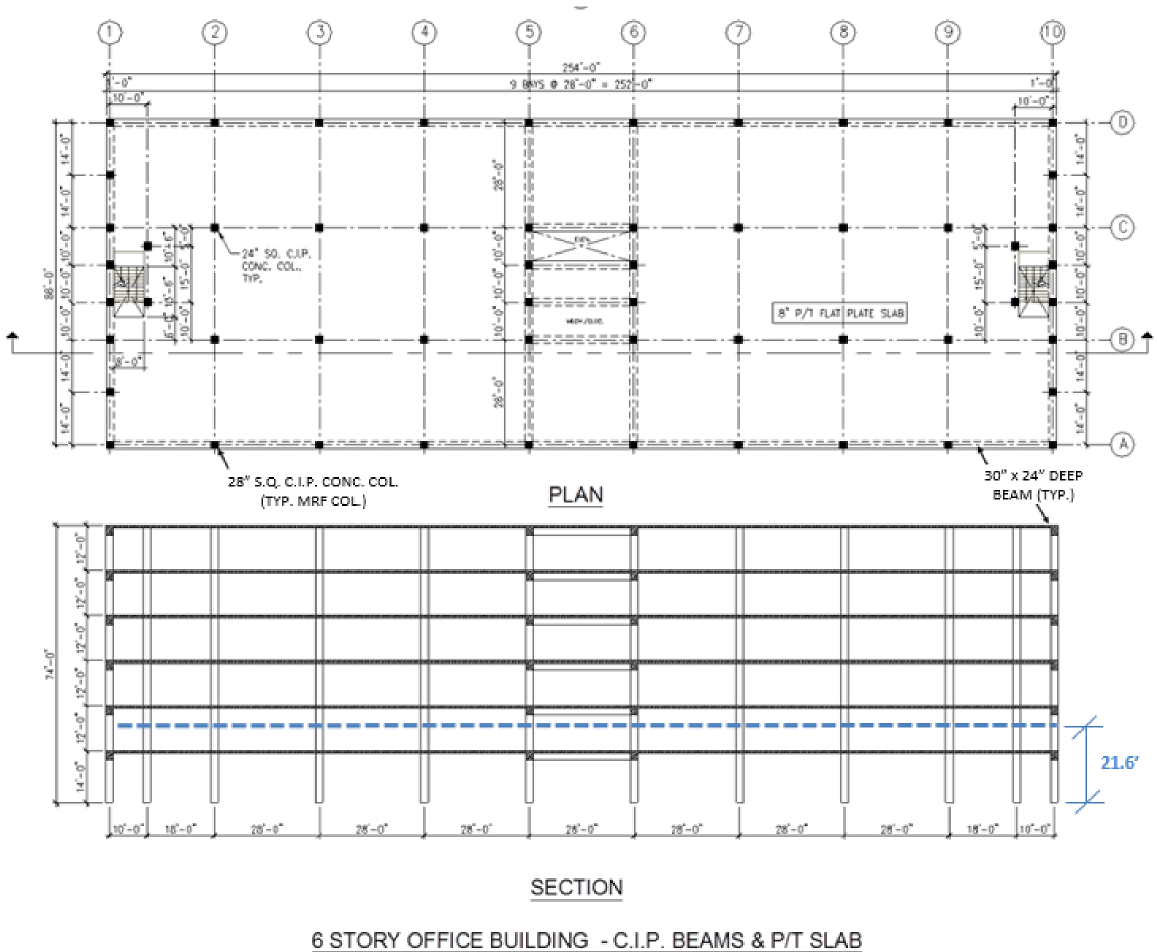

Figure B-15: 6 Story Office Building using Special Reinforced Concrete Moment Frame and posttensioned flat slab supported on gravity columns

B.6.2 7-Story Residential Building

The 7-story residential building consists of a Building Frame System with special reinforced concrete shear walls at exit stairs and elevator core, and interior gravity columns supporting posttensioned floor slabs (See **Figure B-16**). The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: D

Response Spectrum Parameters: S_s = 1.513, S_1 = 0.554, S_{DS} = 1.009, S_{D1} = 0.554

Structural System Response Factors: R = 6, Ω_o = 2.5, C_d = 5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 66 ft. With a maximum flow depth of 21.6 ft, this building could function as a "Refuge of Last Resort" at the 4th level (30 ft) up to the roof (if acceptable).

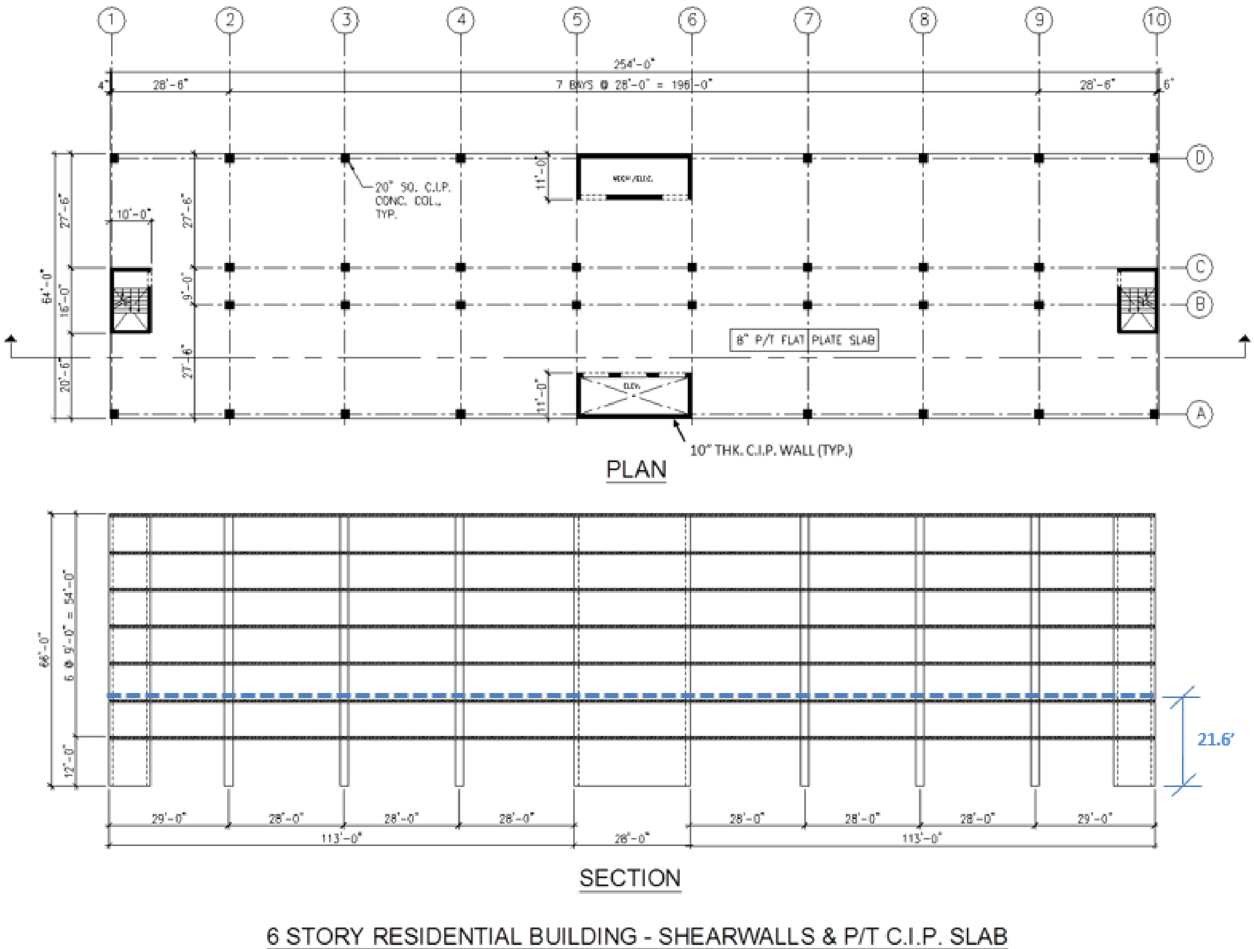

Figure B-16: 7 Story Residential Building using Special Reinforced Concrete Shear Walls and posttensioned flat slab supported on gravity columns

B.7 Tsunami Loading Summary

Table B-3 gives a summary of the tsunami loads determined for the building located at the selected site. This section of this example shows detailed calculation of the tsunami loads, along with evaluation of the structural system and components for these loads. Note that these calculations are far more detailed than would be necessary for a typical design project because the intent here is to provide a complete explanation of the various calculations and their application.

Table B-3: Summary of Tsunami Loading for Office and Residential Buildings

Flow Parameters	Office Building	Residential Building
Max. Inundation Depth, h_{max} (ft)	21.6	21.6
Max. Flow Velocity, u_{max} (fps)	31.5	31.5
Overall Building Lateral Loading (kips)		
Load Case 1	2,281	2,281
Load Case 2	3,678	3,679
Load Case 3	582	3582
Component Loading (kips)		
Exterior Column Hydrodynamic Drag	616 [2]	616 [2]
Interior Column Hydrodynamic Drag	62.9	52.4
Exterior Column Debris Impact	107.25 [3]	107.25 [3]
Exterior Wall Debris Impact	-	107.25 [3]
Wall and Slab Loading (psf)		
Hydrodynamic Pressure on Walls	-	2,660
Stagnation Pressure in Mech/Elec Rm	-	1,091 [5]
Surge Uplift on Elevated Slabs	-	20

[1] Including effect of debris damming, C_{cx}, applied to column tributary width.

[2] Limited by log crushing capacity.

[3] Stagnation pressure acting outwards on structural walls and floor slab enclosing Mech/Elec room corresponding to the maximum velocity and corresponding flow depth.

B.8 Assumed Conditions

The following conditions are assumed to apply for this example:

1. The building is oriented with the longitudinal axis parallel to the shoreline.
2. The building has no basement.
3. The foundation system consists of deep piles with pile caps supporting all shear walls and all exterior columns.
4. The ground floor slab-on-grade has isolation joints at all columns, structural walls and grade beams.
5. The top of the first floor windows is 8 feet above grade, with the window sill at 3 ft.

6. The building location is not in the vicinity of a shipping container storage yard or port facility, and is therefore not subject to debris impact from shipping containers, ships or barges.
7. The non-structural exterior cladding spans vertically between floors.

B.9 Tsunami Design for Office Building

B.9.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

In lieu of performing detailed hydrostatic and hydrodynamic analysis, **Eqn. 6.10.1-1** provides a simplified but conservative estimate of the maximum lateral load on the building.

$$\mathrm{f_{uw}} = 2.5\mathrm{I_{tsu}}\gamma_{\mathrm{s}}\mathrm{h}_{\max}^2 = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 21.6^2 = 82.1\ \mathrm{kip/ft}$$

Assuming $\mathrm{C_{cx}} = 0.7$ controls per **Section 6.8.7**

Then $\quad \mathrm{F} = 0.7 \times 254 \times 82.1 = 14{,}600\ \mathrm{kips}$

This lateral load can be compared with 0.75 Ω_o E_h = 0.75 x 3 x 2,259 = 5,083 kips < 14,600 kips. The detailed analysis for LC2 and LC3 should therefore be performed as shown above, in which case the LFRS is adequate with no strengthening.

B.9.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

B.9.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} $\leq h_{max}$ = 21.6 ft

$\leq$ 14 ft (ground floor story height)

$\leq$ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

For the sake of illustration, if we had assumed that the ground floor consists of structural grade beams and integral slab on grade without isolation joints, and that the soil allowed ground water pressure increase below the building (i.e. sandy or gravely subsoil), the buoyancy would need to be considered as follows:*

Section 6.9.1, Eqn. 6.9.1-1 $F_v = \gamma_s V_w$ = (1.1x64.0)(254’ x 88’ x 8’)/1000 = 12,588 kips

Apply load combination: 0.9D + F_{TSU} + 1.2 H_{TSU}

where H_{TSU} = 0 *since scour is assumed uniform around the building perimeter*.

and building dead weight, D = 16,000 kips, *including foundation*.

Therefore net uplift = - 0.9 x 16,000 + 12,588 = -1812 kips, downward.

Overall uplift would therefore not be a concern, even if the ground floor were a structural slab capable of resisting the associated buoyancy pressures. This example also ignores any uplift resistance provided by the deep foundations.

In combination with buoyancy, Load Case 1 requires application of the associated hydrodynamic drag on the entire building.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254' overall width of building

h = 8'

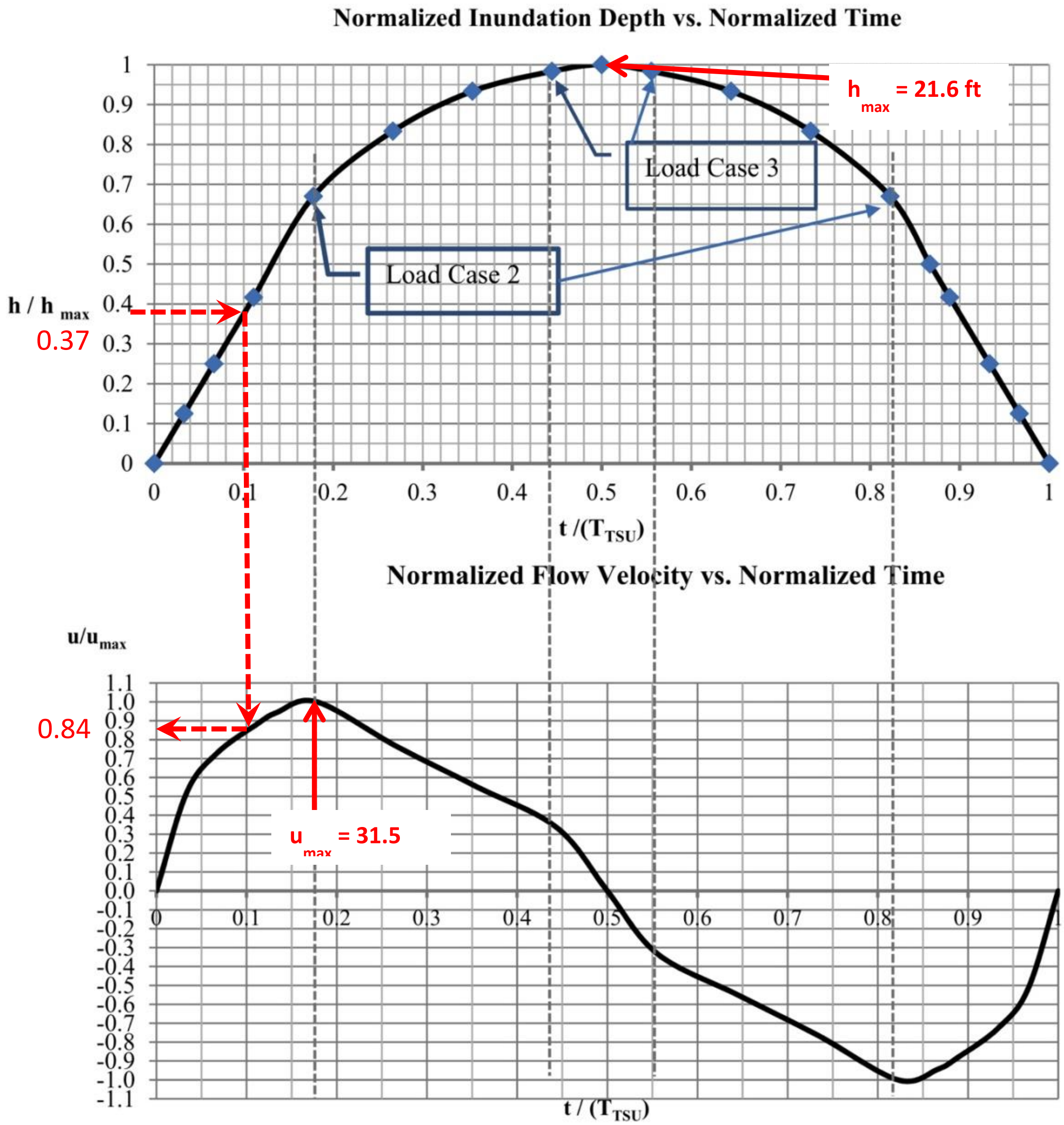

Figure B-17: Determining "u" for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/21.6 = 0.37. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.11. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.84. Therefore the flow velocity is u = 0.84 x 31.5 = 26.46 fps.

So$F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 26.46^2)/1000 = 2{,}281\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure at the

first floor level would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail so as to relieve lateral load on the structural frame. Note that only portion of this load will go to the second floor slab, which therefore has to be resisted by the lateral force resisting system. The majority of the load will go directly to the grade beam/foundation system. The entire lateral load must be resisted by the deep foundation assuming maximum scour has already occurred.

B.9.2.2 Load Case 2: Maximum Flow Velocity

In this particular example, LC1 and LC2 are very similar for the overall building, but the following calculation is shown for completeness.

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is $2/3h_{max}$ = 2/3 x 21.6 = 14.4 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.316 (**Table 6.10-1** based on B/h_{sx} = 254/14.4 = 17.64)

Since the inundation depth of 14.4 feet exceeds the bottom of the second floor beams (14' – 24"/12) = 12', the inundated area of the beams must be included in the closure coefficient, which is given by:

h_{sx} = 14.4 ft.

$h_{col\ EQ}$ = 14.4'-24"= 12.4' (Clear height of submerged Moment Resisting Frame columns)

$A_{col\ EQ}$= 12.4'x2.33'x40 = 1,156 ft^2 (40 MRF earthquake columns each 28" wide)

$h_{col\ Grv}$= 14.4'-8"= 13.73' (clear height of submerged gravity load columns)

$A_{col\ Grv}$= 13.73'x2'x16 = 439 ft^2 (16 gravity load column, each 2' wide)

A_{wall} = 0 ft^2 (no walls in MRF structure)

A_{beam} = 24"x254'x1= 508 ft^2 (1x24" deep beam goes above the 2nd level beam)

$$C_{cx} = \frac{\sum(A_{col} + A_{wall}) + 1.5A_{beam}}{Bh_{sx}} = \frac{\sum((1156 + 439) + 0) + 1.5 \times 508}{254' \times 14.4'} = 0.644 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

$u = u_{max}$ = 31.5 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.316 \times 0.7 \times 254(14.4 \times 31.5^2)/1000 =$
$3{,}678\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal or inland elevation of the building over a height of 14.4 feet above grade as shown in **Figure B-18**. The lateral force resisting system for the structure at the first and second floor levels would be evaluated for this load.

B.9.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 21.6 ft. and the flow velocity is $1/3u_{max}$ = 1/3 x 31.5 = 10.5 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/21.6 = 11.8)

Since the inundation depth of 21.6 feet exceeds the bottom of the second floor beams (14' – 24") = 12', the inundated area of the second floor beams must be included in the closure coefficient, which is given by:

h_{sx} = 21.6 ft.

$h_{col\ EQ}$ = 21.6'-24"= 19.6'

$A_{col\ EQ}$= 21.6'x2.33'x40 = 1,829 ft^2

$h_{col\ Grv}$= 21.6'-8"= 20.93'

$A_{col\ Grv}$= 20.93'x2'x16 = 669 ft^2

A_{wall} = 0 ft^2

A_{beam} = 24"x254'x1= 508 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum((1829+669)+0)+1.5\times 508}{254\prime \times 21.6\prime} = 0.594 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 10.5 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(21.6 \times 10.5^2)/1000 = 582\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 21.6 feet above grade as shown in **Figure B-19**. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth for hydrodynamic load and debris impact.

B.9.3 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category D, **Section 6.8.3.4** permits the use of $0.75\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear. From the seismic design of this structure, E_h = 2,259 kips. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 3 \times 2{,}259 = 5{,}083\ kips$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 3,679 kips applied over a height of 14.4 ft. A portion of this load will be resisted by the grade beam/foundation system as shown in **Figure B-18**, reducing the overall load by 1,788 kips. Therefore, V_{TSU} = 3,678 – 1,788 = 1,890 kips. Applying the LFRS assessment gives:

$$0.75\Omega_o E_h = 5{,}083\ kips > 1{,}890\ kips \quad \therefore OK$$

So the lateral force resisting system has ample capacity to resist the overall tsunami loads. In order to combine these systemic effects with the individual component loads on each member of the lateral force resisting system, the building must be analyzed for a seismic base shear, E_h, of:

$$E_h = \frac{V_{TSU}}{0.75\Omega_o} = \frac{1{,}890}{0.75 \times 3} = 840\ kips$$

This seismic base shear must be distributed up the height of the building following ASCE 7 seismic design provisions. This Load Case 2 base shear of 840 kips was applied to the same ETABS model used for the original wind and seismic analysis of the building. Load Case 3 was also analyzed but did not control any of the member designs. This ETABS analysis resulted in the column forces shown in **Figure B-20** and **Figure B-21** for floors one and two, respectively.

While acting as part of the lateral force resisting system, these columns are also subjected to component drag or debris impact loads. According to ASCE 7 Section 6.8.3.5, the columns in the inundated floors must be designed and detailed for these higher forces "*that result from the overall tsunami forces on the structural system combined with any resultant actions caused by the tsunami pressures acting locally on the individual structural components for that direction of flow*". All members of the LFRS must resist the forces resulting from the overall system analysis, in combination with hydrodynamic and impact loads acting on the member itself.

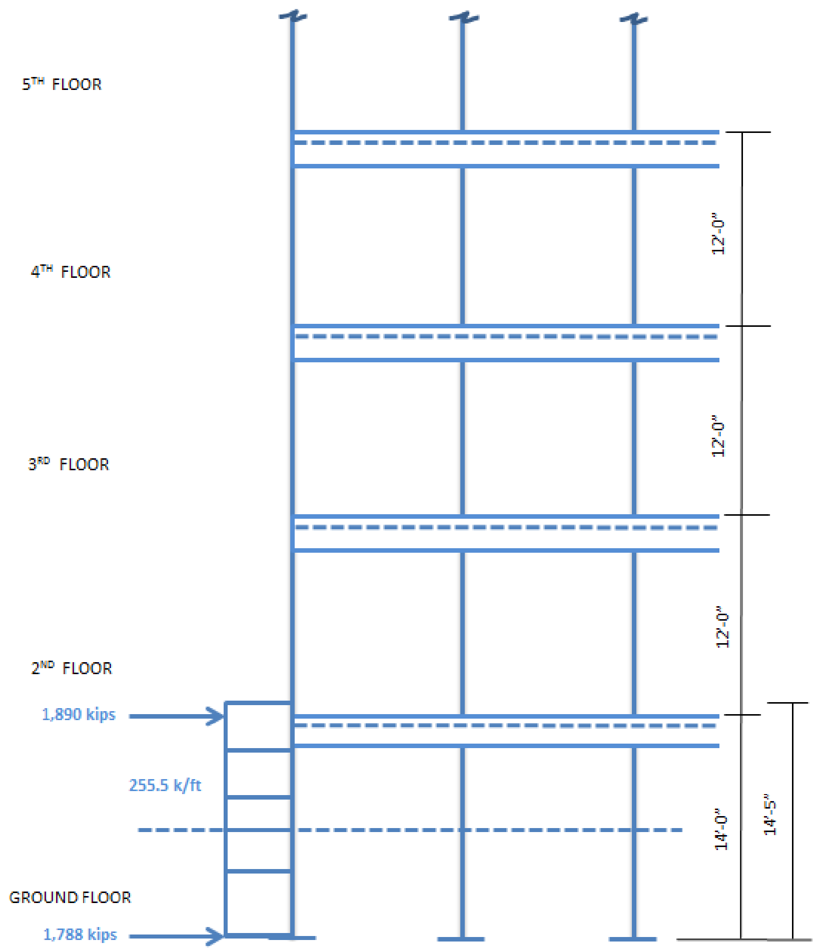

Figure B-18: LC2 Tsunami loads on overall Monterey office building

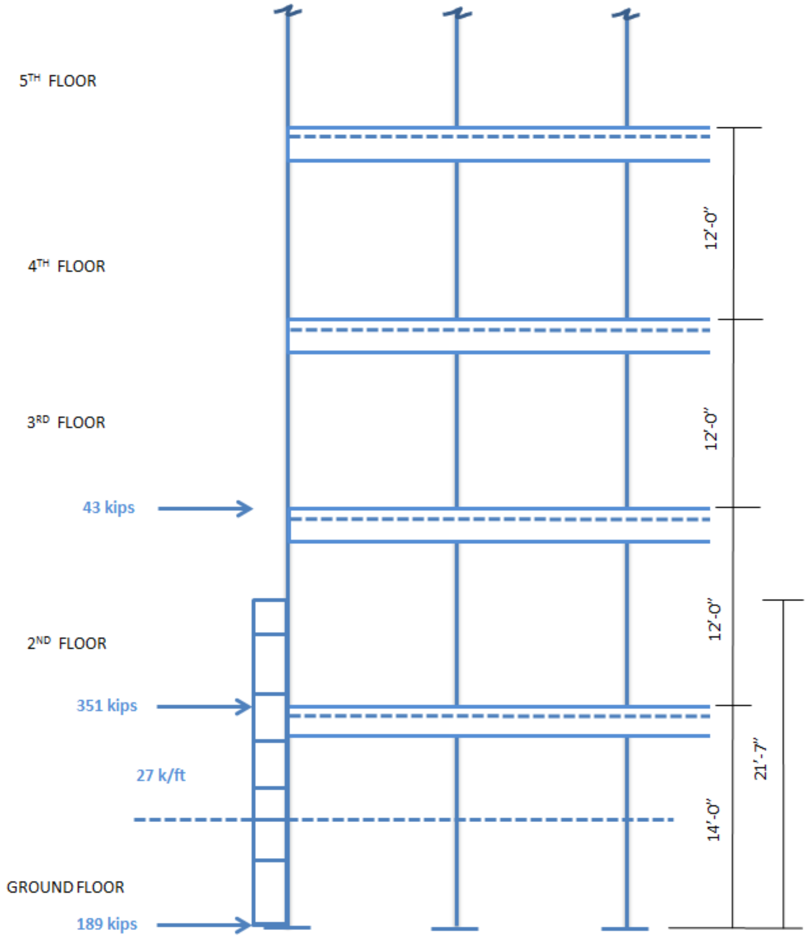

Figure B-19: LC3 Tsunami loads on overall Monterey office building

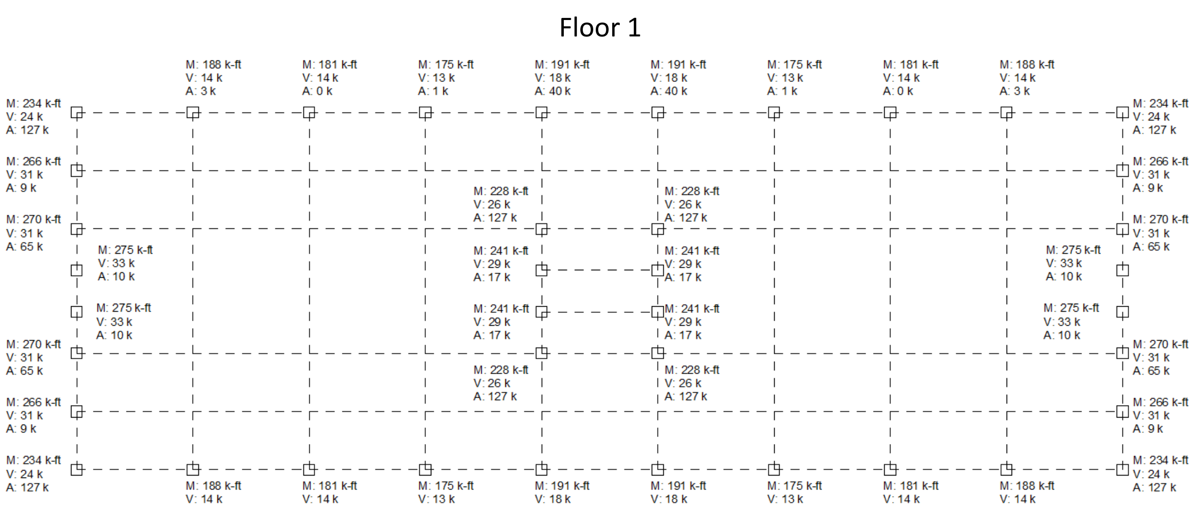

Figure B-20: Maximum forces in the first floor columns due to tsunami base shear

Floor 2

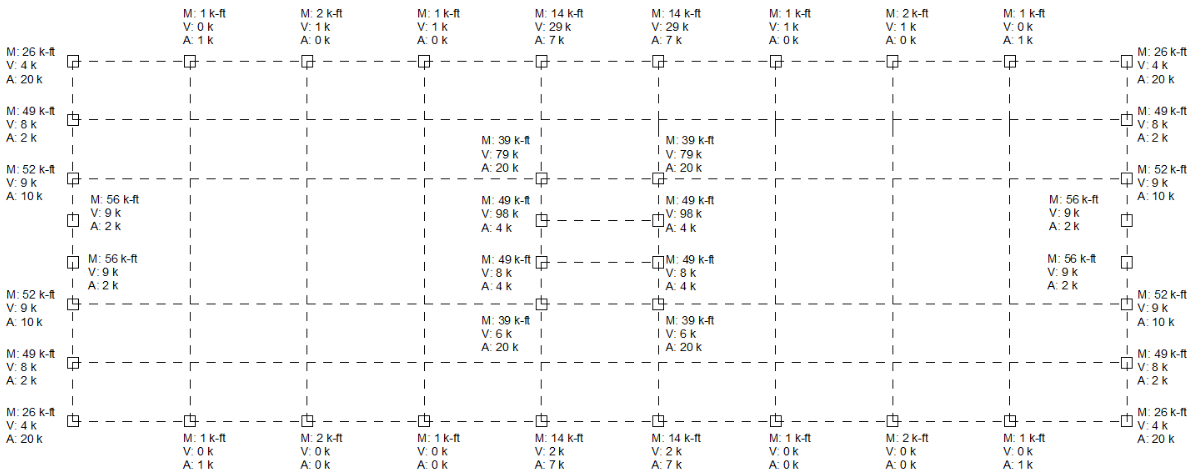

Figure B-21: Maximum forces in the second floor columns due to tsunami base shear

B.10 Component Design

B.10.1 Drag Force on Components - Section 6.10.2.2

B.10.1.1 Exterior Columns

For Load Case 1, the exterior cladding is assumed to remain intact. Since the cladding spans vertically between floors for this example building, none of the hydrodynamic lateral load in LC1 will be applied directly to the ground floor columns. [*Note that if the exterior cladding were supported by girts which transferred lateral load to the columns, then the columns would need to be designed for this load*.]

For Load Cases 2 and 3, the exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** will require that C_d = 2.0 and the width dimension, *b*, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Previous calculation of C_{cx} showed that the default value of 0.7 controls for LC2 and LC3 for this building, Therefore *b* = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 14.4 ft and u_{max} = 31.5 fps.

The hydrodynamic drag is computed using **Eqn 6.10-4** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(14.4 \times 31.5^2)/1000 = 616\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 616/14.4 = 42.8 kips/ft over the lower 14.4 feet of the column. The column must be designed for this load combined with gravity loads using the load combinations in **Section 6.8.3.3**. In addition, because the exterior columns are part of the LFRS, these component loads must be combined with the systemic forces and the column designed for the combined loads.

B.10.1.2 Interior Columns

Interior columns are 24" (2 ft) square R.C. columns. For Load Case 1, the interior is not yet inundated, so there are no hydrodynamic loads on the interior columns. The controlling load case will be LC2, when the inundation depth is h_e = 14.4 ft and u_{max} = 31.5 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**)

Therefore $$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 2.0(14.4 \times 31.5^2)/1000 = 62.9\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 62.9/14.4 = 4.37 kips/ft over the lower 14.4 feet of the column. This load must be combined with gravity loads using the load combinations in **Section 6.8.3.3** and the column capacity verified.

B.10.2 Other Hydrodynamic Loads

No other hydrodynamic load conditions apply to this building since there are no structural walls and the spandrel beam is integral with the slab so the lateral load on the beam will transfers directly to the slab diaphragm.

B.10.3 Debris Impact Loads - Section 6.11

The inundation depth at the site exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads per **Section 6.11**.

B.10.3.1 Detailed Debris Impact Calculation for Office Building

Wood Logs and Poles - Section 6.11.2

The nominal maximum instantaneous debris impact force is given by **Eqn. 6.11-2** as:

$$F_{ni} = u_{max}\sqrt{km_d}$$

Where u_{max} = 31.5 fps

$k = EA/L$ for the wood log with a minimum value of 350 k/in ($4.2\text{x}10^6$ lb/ft)

m_d = 1000/32.2 = 31.1 slugs for the minimum 1000 lb log.

Therefore: $$F_{ni} = u_{max}\sqrt{km_d} = 31.5\sqrt{4.2 \times 10^6 \times 31.1}/1000 = 360\ kips$$

The design instantaneous debris impact force is then given by **Eqn. 6.11-3** as:

$$F_i = I_{tsu} C_0 F_{ni} = 1.0\ \times 0.65 \times 360 = 234\ kips$$

The impulse duration is given by **Eqn. 6.11-4** as:

$$t_d = \frac{2m_d u_{max}}{F_{ni}} = \frac{2\times31.1\times31.5}{360{,}000} = 0.0054\ sec$$

The column can be designed using a dynamic analysis by applying an impulsive rectangular pulse with magnitude F_i and duration t_d. Alternatively an equivalent elastic static analysis can be performed of the column subjected to F_i multiplied by a dynamic response factor, R_{max}, given in **Table 6.11-1**. The ratio of impact duration to natural period of the impacted structural element is obtained using t_d and the natural period of the column assumed to be fixed-fixed. For this case, the natural period is given by;

$$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}}$$

Where L = unbraced column length = 14' – 24"/12 = 12 ft for the ground floor columns.

ρ = column mass per unit length = 2.333'x2.333'x150pcf/32.2ft/s^2 = 25.36 slugs/ft

E = modulus of elasticity of the column concrete = 3600 ksi = 518.4x10^6 psf

I = moment of inertia of column section = $bd^3/12$ = 2.333x2.333^3/12 = 2.47 ft^4

Therefore $$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}} = 2\pi\left[\frac{12^2}{22.373}\right]\sqrt{\frac{25.36}{518.4\times10^6\times2.47}} = 0.00444\ sec$$

The ratio of impact duration to column natural period is therefore t_d/T_{col} = 0.0054/0.00444 = 1.22.

Table 6.11-1 gives the dynamic response factor R_{max} = 1.6, therefore the equivalent static load is given by;

$$F_{es} = R_{max}F_i = 1.6\times234 = 374\ kips.$$

This exceeds the maximum required impact force of 107.25 kips, therefore the column can be evaluated for a lateral point load of 107.25 kips applied at locations which are critical for flexure and shear.

B.10.4 Impact by Vehicles – Section 6.11.3

The impact force is given as $F_i = I_{tsu}$ x 30 = 30 kips. This will not control over the log impact load determined above.

B.10.5 Impact by Submerged Tumbling Boulder and Concrete Debris – Section 6.11.4

Because h_{max} = 21.6 ft > 6 ft, an impact force of $F_i = I_{tsu}$ x 8 = 8 kips shall be applied at 2ft above grade. This will not control over the log impact load determined above.

B.10.5.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330C_0I_{tsu} = 330\times0.65\times1.0 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force will be reducible to 50%, or 107.25 kips. This load must be applied to the exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with hydrodynamic loads on the column, but it must be combined with systemic loads if the member is part of the lateral force resisting system. In the event that this load exceeds the column capacity, a detailed debris impact analysis can be performed. Debris impact loads are not applied to interior columns.

B.11 Column Design for Tsunami Loads

B.11.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 from **Figure B-15**. The column is part of the lateral force resisting system for longitudinal seismic load designed and detailed for Seismic Design Category D. The column has been designed for gravity and seismic loads resulting in the cross-section shown in **Figure B-22** and **Figure B-23** at the ground floor level and **Figure B-24** and **Figure B-25** for the remaining floor levels. The column will now be checked for tsunami load combinations.

Seismic design of the columns requires additional column ties to ensure ductility of the yield zones at each end of the column. These zones have a length equal to the maximum column cross-section dimension, in this case 28 inches. The critical shear force in this yielding zone occurs at a distance "*d*" from the top and bottom of the column, where *d* = 28 - 1.5 - 0.5 - 0.635 = 25.365 in. The critical shear force for the internal section of the column occurs at "d + h" from the edge of the column, where d + h = 25.365 + 28 = 53.365 in. The column ties required for seismic design will be evaluated for the shears induced by the tsunami both in the end section and center section of the column (**Figure B-26**).

Floor 1

End Section (A)

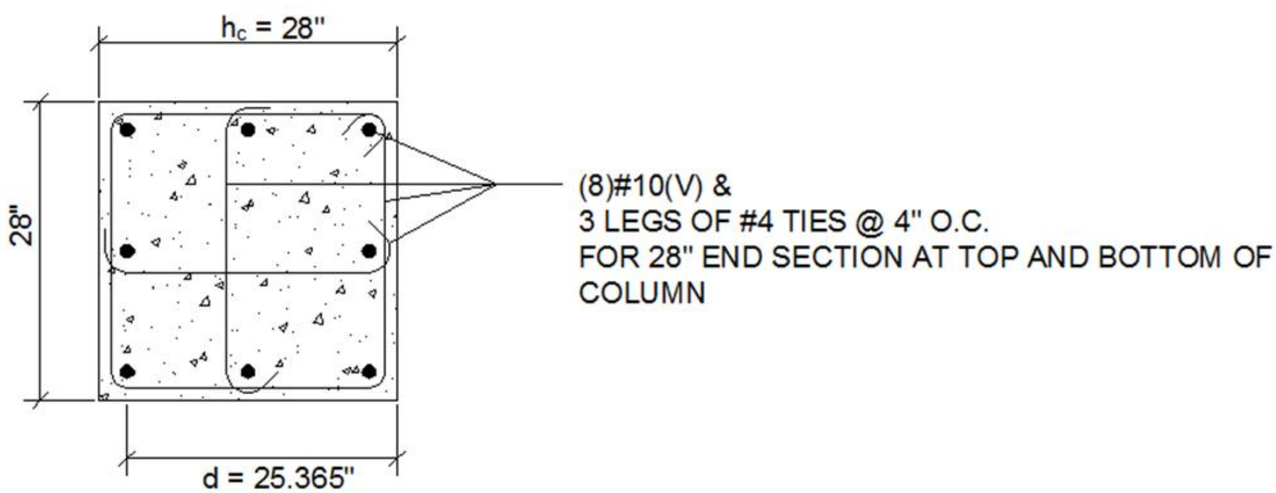

Figure B-22: Exterior column, cross-section at end of column at ground floor level based on SDC D design.

Center Section (B)

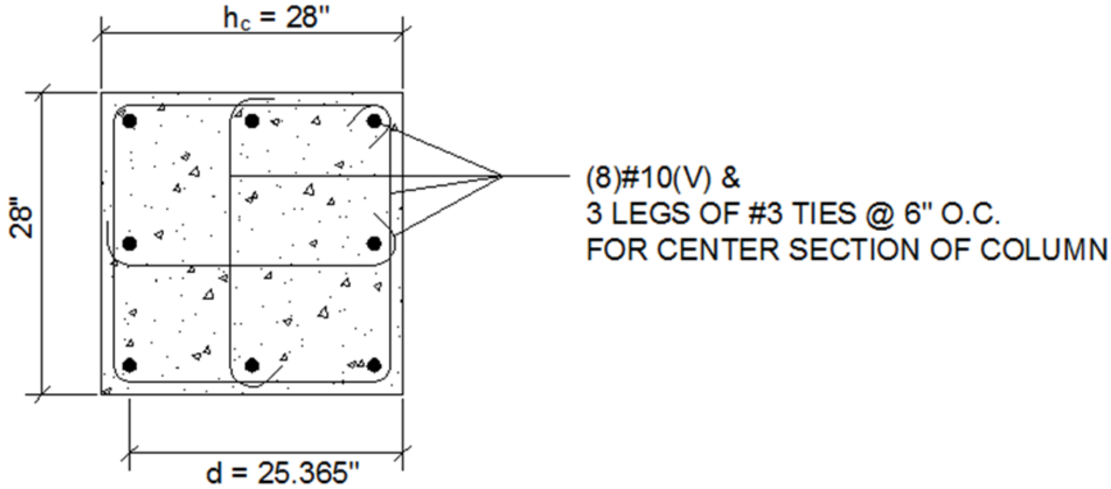

Figure B-23: Exterior column, cross-section at center of column at ground floor level based on SDC D design.

Floor 2 – 6

End Section (A)

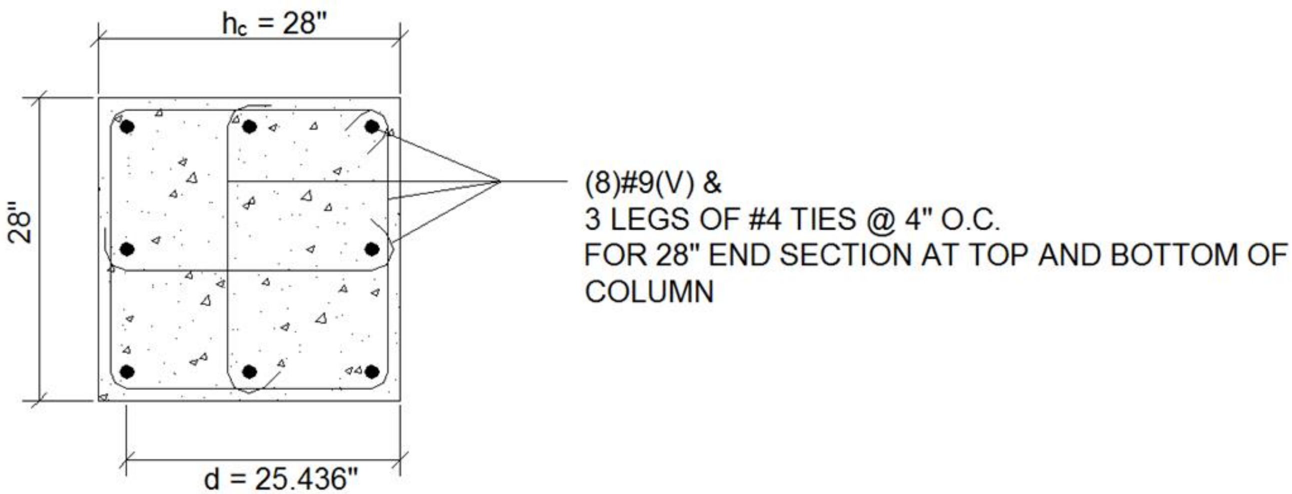

Figure B-24: Exterior column, cross-section at end of column at 2nd – 6th floor levels based on SDC D design.

Center Section (B)

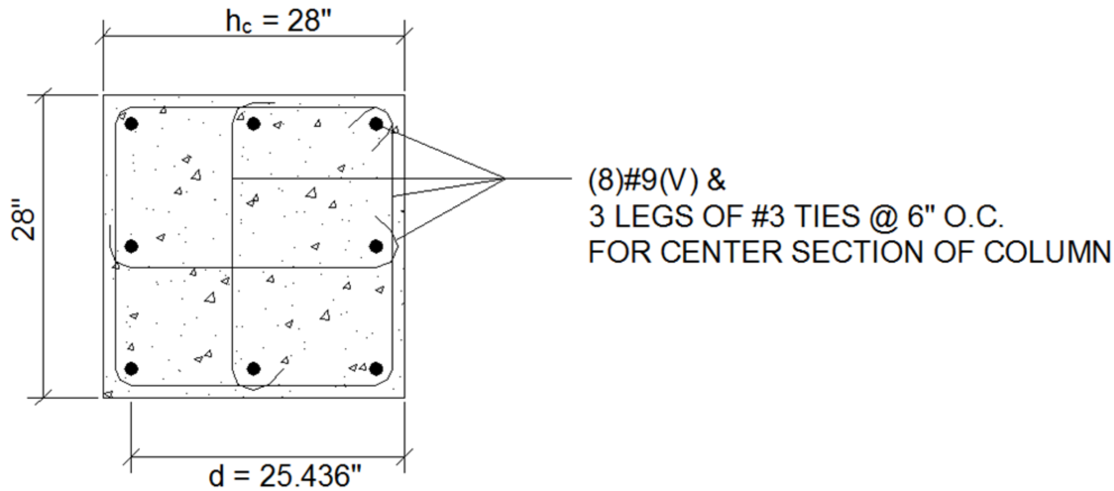

Figure B-25: Exterior column, cross-section at center of column at 2nd – 6th floor levels based on SDC D design.

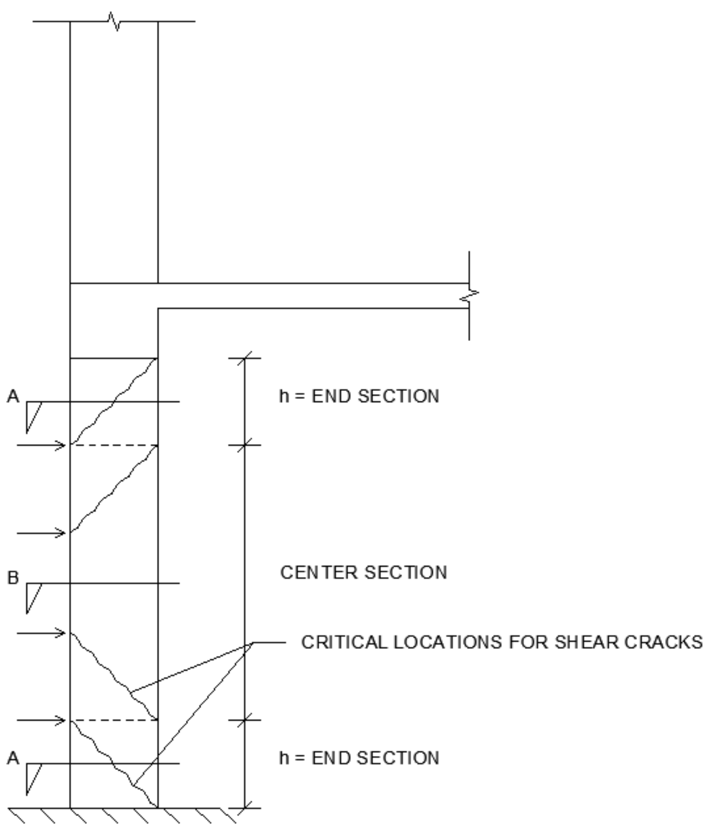

Figure B-26: Typical exterior column elevation showing end and center sections

B.11.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 15 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

P_D = [120(28)(15)(6)+(1.16)(2.5)(150)(28-2.5)+90(28)(5)+2.5^2(150)(74)]/1000 = 395 k

Floor Live load reduction factor = $0.25+15/[4(15)(28)(5)]^{0.5} = 0.414$,

therefore, live load at the column base is: P_L = 0.414[65(15)(28)(5)]/1000 = 56.5 k

Roof Live Load reduction factor = R_1R_2= [1.2-(0.001)(15)(28)](1.0) = 0.78,

therefore, column roof live load is: P_{Lr} = 0.78(20)(15)(28)/1000 = 6.55k

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

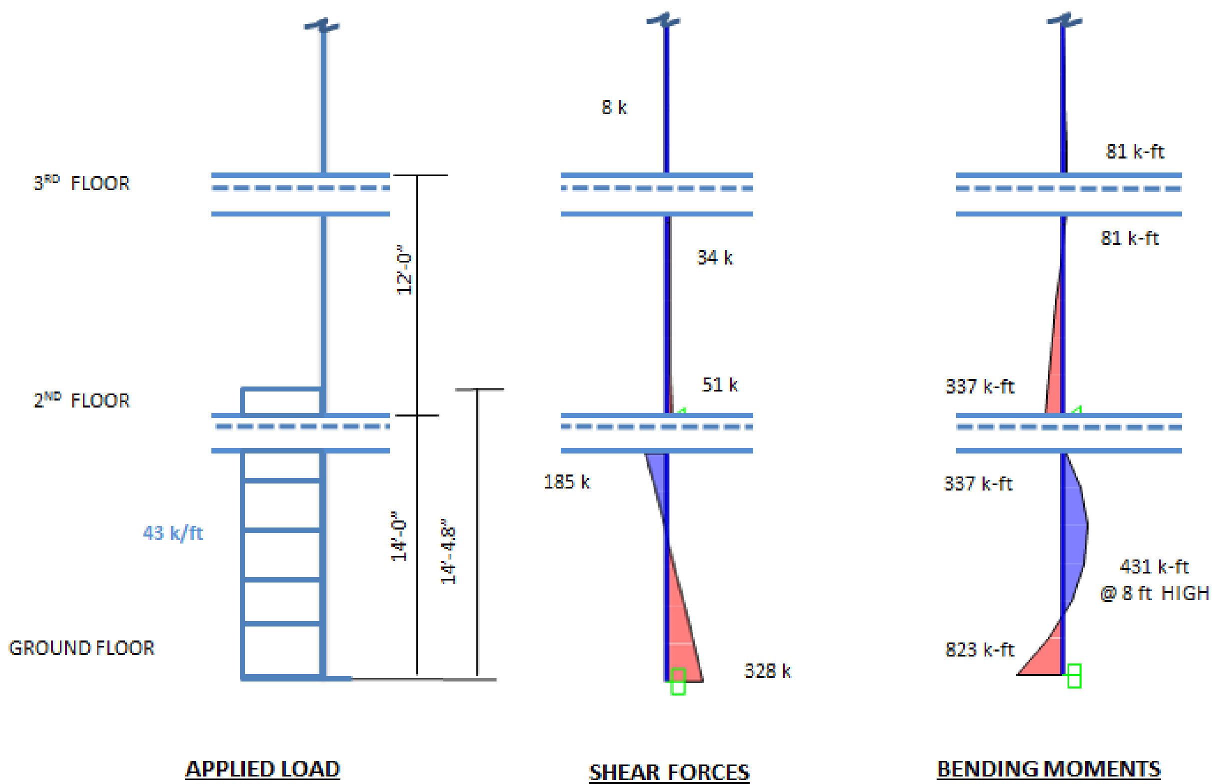

Figure B-27: Hydrodynamic loading on exterior column of the Monterey office building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just below the beam at each inundated floor for the maximum shear in the end section of the column. A log strike is also assumed to act just outside the end section (at "d + h_c") and at the mid-height of the clear column height for the maximum shear force and bending moment in the center section, respectively. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure B-28** to **Figure B-29**. The resulting shear force and bending moment diagrams for log impact at a distance "d + h_c" from the end of the column at each floor level are shown in **Figure B-30** to **Figure B-31**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown in **Figure B-32** to **Figure B-33**.

Impact load at d:

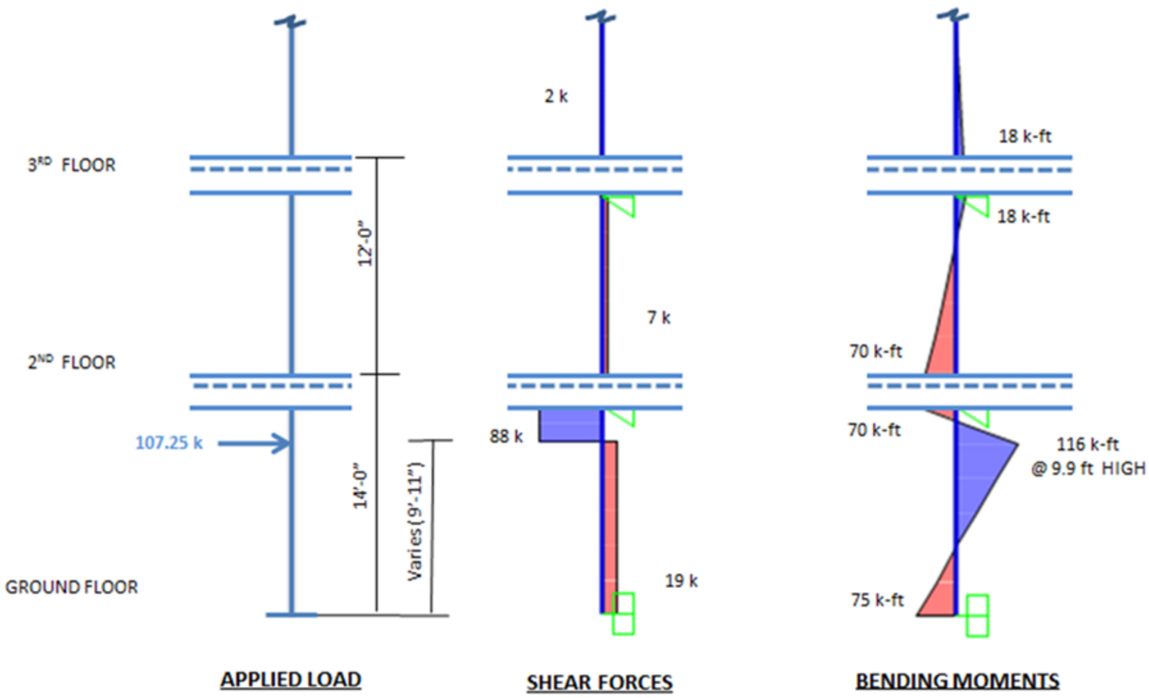

Figure B-28: Impact load applied at "d" away from the end of column on the ground floor

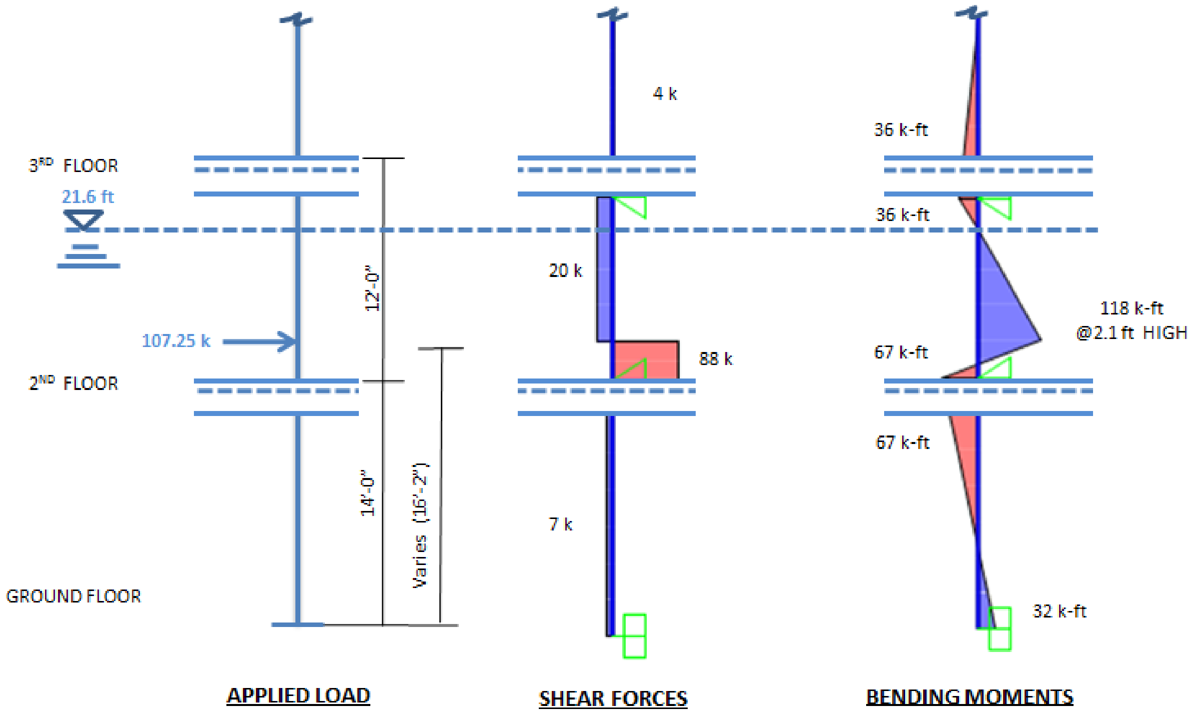

Figure B-29: Impact load applied at "d" away from the end of column on the 2nd floor

Impact load at d + h_c:

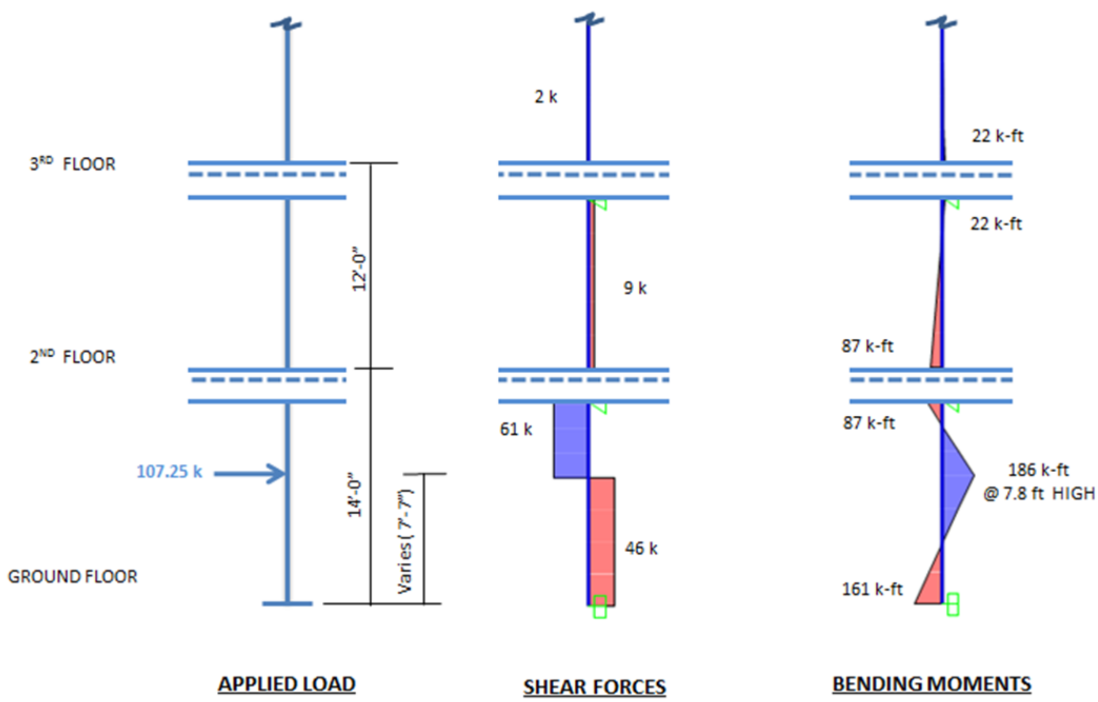

Figure B-30: Impact load applied at "d + h_c" away from the end of column on the ground floor

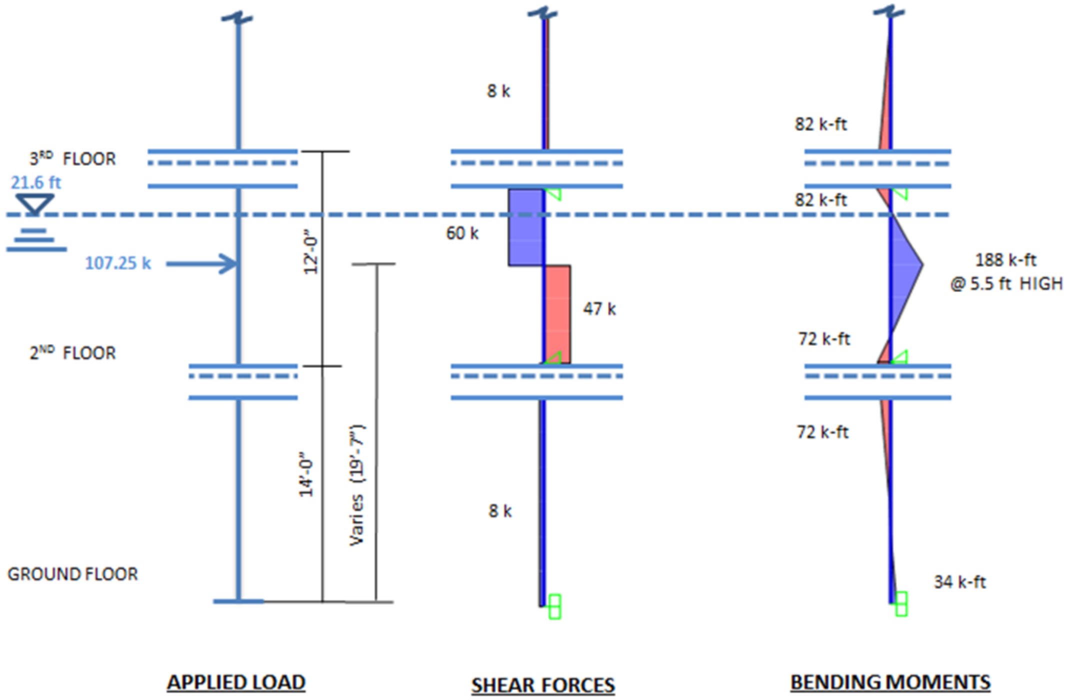

Figure B-31: Impact load applied at "d + h_c" away from the end of column on the 2nd floor

Impact load at mid-height:

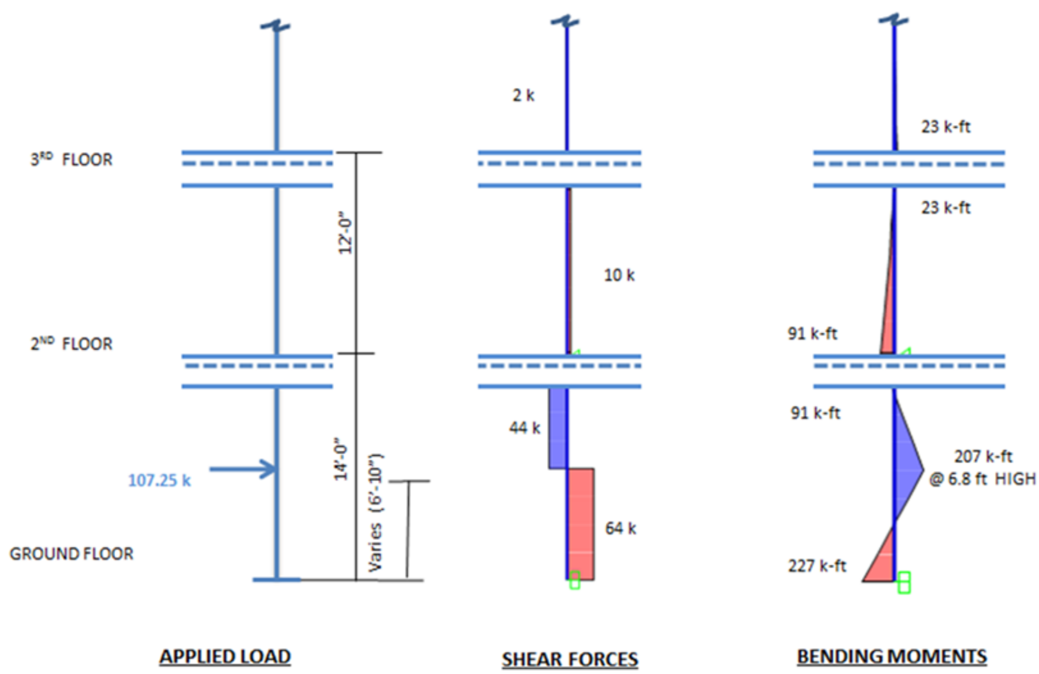

Figure B-32: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

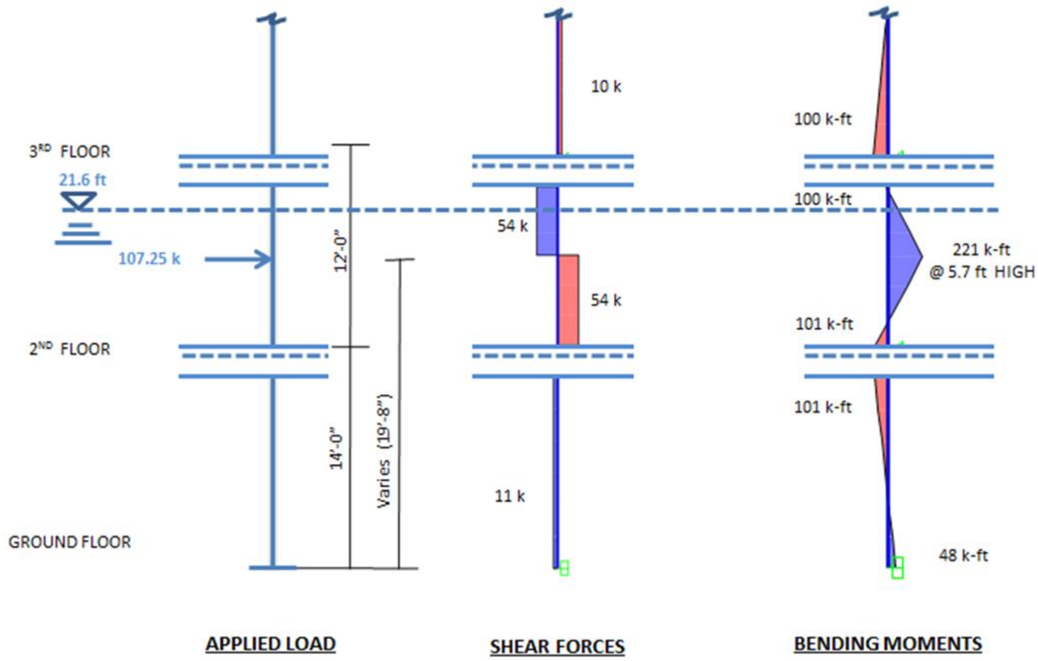

Figure B-33: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

Table B-4 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and log impact (Impact). In addition, because all of the exterior columns are part of the LFRS, **Table B-4** also lists the maximum axial load, bending moment and shear forces determined by the ETABS analysis for the modified base shear (Overall) (See Section A.9.3). These "Overall" systemic forces are then combined with the controlling component forces (either "Hydro" or "Impact") to obtain the "Combined" forces. Columns that are part of the transverse MRFs experience larger systemic loads and are therefore considered separately, along with columns having similar loads ("Special").

The original column designs will now be evaluated for these load combinations and modified if necessary.

Table B-4: Results from loading conditions of Monterey office building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
823	502.3	238	138	1.2D+Ftsu+0.5L (Hydro)
823	355.5	238	138	0.9D+Ftsu (Hydro)
227	502.3	88	61	1.2D+Ftsu+0.5L (Impact)
227	355.5	88	61	0.9D+Ftsu (Impact)
275	492.3	127	127	1.2D+Ftsu+0.5L (Overall)
275	345.5	127	127	0.9D+Ftsu (Overall)
1014	462.3	365	265	1.2D+Ftsu+0.5L (Combined)
1014	315.5	365	265	0.9D+Ftsu (Combined)
Floor 2				
337	418.5	34	34	1.2D+Ftsu+0.5L (Hydro)
337	296.3	34	34	0.9D+Ftsu (Hydro)
221	418.5	88	60	1.2D+Ftsu+0.5L (Impact)
221	296.3	88	60	0.9D+Ftsu (Impact)
56	416.5	9	9	1.2D+Ftsu+0.5L (Overall)
56	294.3	9	9	0.9D+Ftsu (Overall)
351	411.5	97	69	1.2D+Ftsu+0.5L (Combined)
351	289.3	97	69	0.9D+Ftsu (Combined)
Floor 3				
81	334.8	8	8	1.2D+Ftsu+0.5L (Hydro)
81	237	8	8	0.9D+Ftsu (Hydro)
100	334.8	10	10	1.2D+Ftsu+0.5L (Impact)
100	237	10	10	0.9D+Ftsu (Impact)
Floor 4				
20	251.1	2	2	1.2D+Ftsu+0.5L (Hydro)
20	177.8	2	2	0.9D+Ftsu (Hydro)
25	251.1	3	3	1.2D+Ftsu+0.5L (Impact)
25	177.8	3	3	0.9D+Ftsu (Impact)
Floor 5				
5	167.4	1	1	1.2D+Ftsu+0.5L (Hydro)
5	118.5	1	1	0.9D+Ftsu (Hydro)
6	167.4	1	1	1.2D+Ftsu+0.5L (Impact)
6	118.5	1	1	0.9D+Ftsu (Impact)
Floor 6				
1	83.7	0	0	1.2D+Ftsu+0.5L (Hydro)
1	59.3	0	0	0.9D+Ftsu (Hydro)
2	83.7	0	0	1.2D+Ftsu+0.5L (Impact)
2	59.3	0	0	0.9D+Ftsu (Impact)

B.11.1.2 Existing Exterior Column Design for Combined Gravity and Tsunami Loads

The column chosen at Grid Intersection A-3 from **Figure A-15** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure B-34 shows the interaction diagram for the typical exterior column including the tsunami load combinations.

The blue solid line (Original Column Design Strength) represents the design strength for the original columns. The green dashed line (New Column Design Strength) represents the design strength needed if one were to only take into account the hydrodynamic and impact loads as well as the design strength needed for taking into account only the overall building forces for each column shown in **Figure B-27** to **Figure B-33**. The orange dot-dashed line (New Combined Column Design Strength) represents the design strength needed for the overall loading combined with the scaled hydrodynamic and impact loads per column. The interaction diagrams for the combined forces shown in every other **Figure B-35** to **Figure B-37** shows the scatter plot of each column and the maximum force is transposed in every other **Figure B-34** to **Figure B-36** to show all of the interaction diagrams for the design strengths needed for each design consideration.

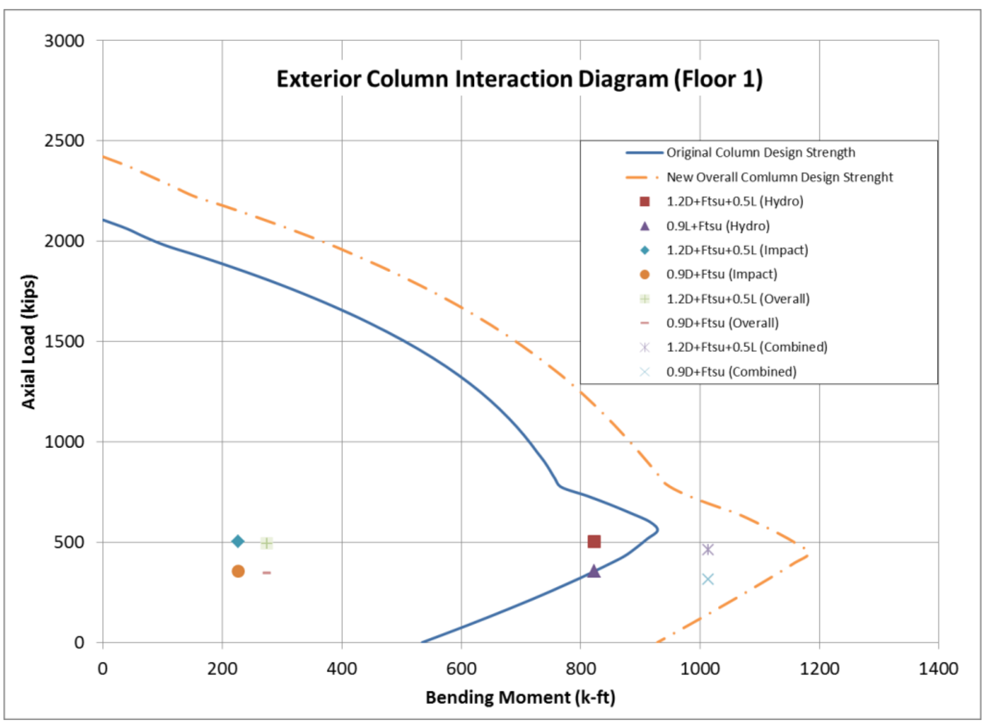

Figure B-34: Sequence of interaction diagrams for typical ground floor exterior column showing tsunami load combinations

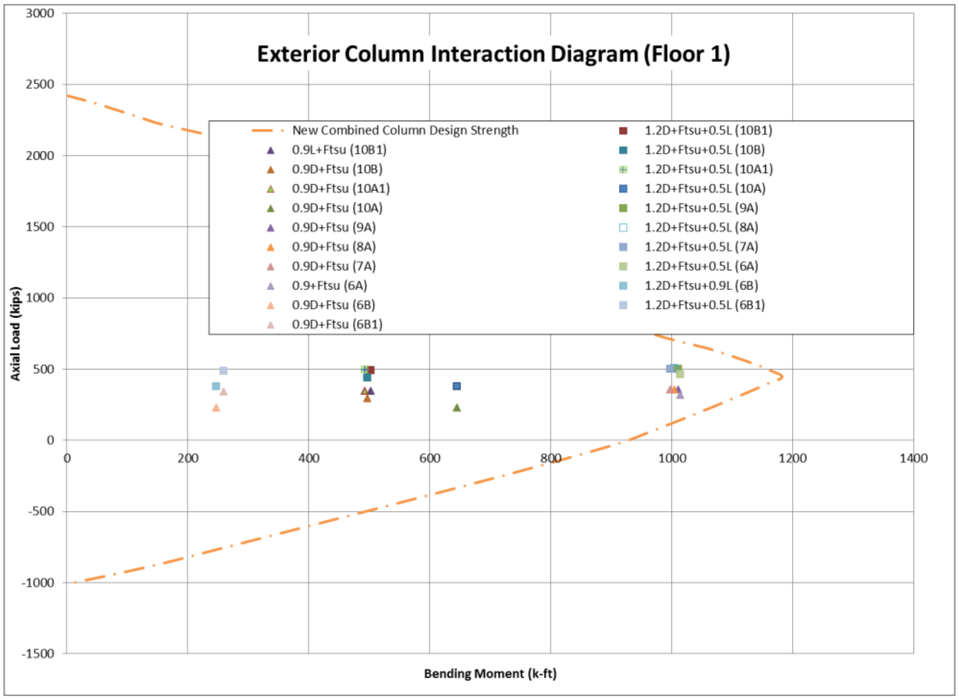

Figure B-35: Interaction diagrams for typical and special ground floor exterior column showing all combined tsunami load combinations

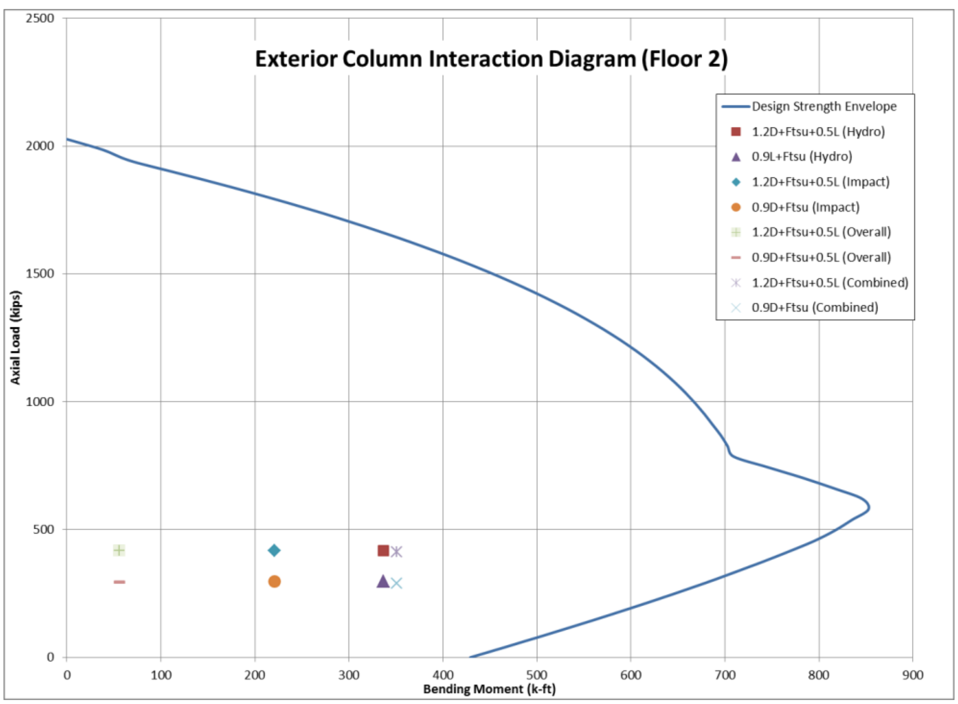

Figure B-36: Sequence of interaction diagrams for typical second floor exterior column showing tsunami load combinations

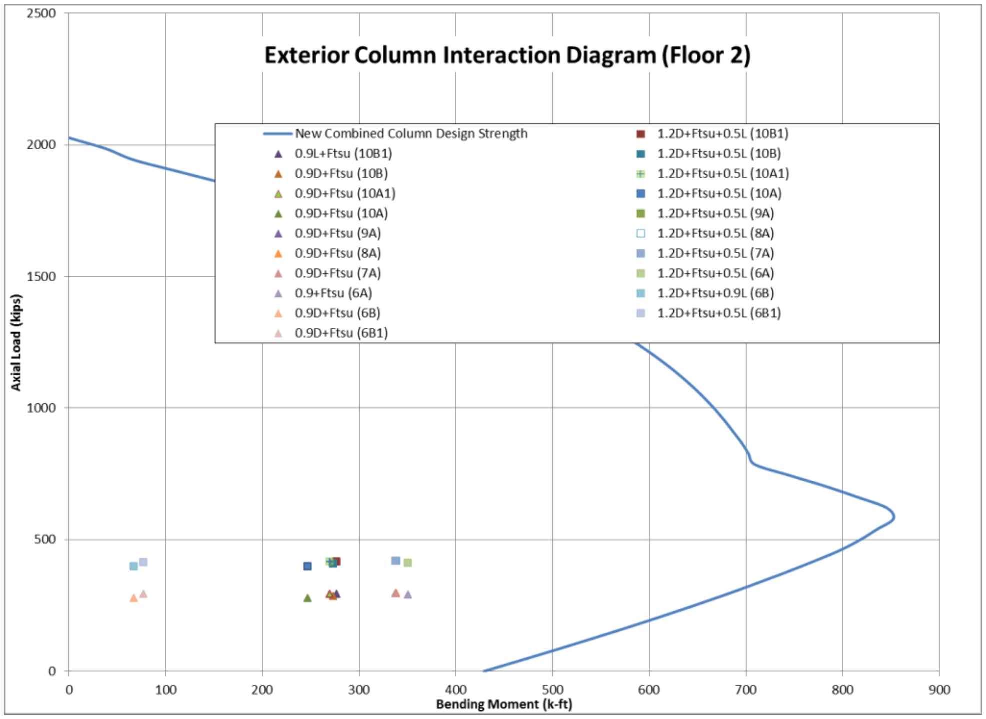

Figure B-37: Interaction diagrams for typical and special second floor exterior column showing all combined tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

B.11.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure B-34** the original exterior columns are adequate for log impact loads, but the columns at the ground floors must be strengthened to resist bending due to the combined hydrodynamic and overall system loads. Revised column designs shown in **Figure B-34** were developed to satisfy the combined hydrodynamic and overall loads. The interaction diagrams for these new columns are shown in Figure C-49 to Figure C-53 The ties in these columns are designed in Section A.11.1.4 for the applied tsunami shear forces.

Floor 1

End Section (A)

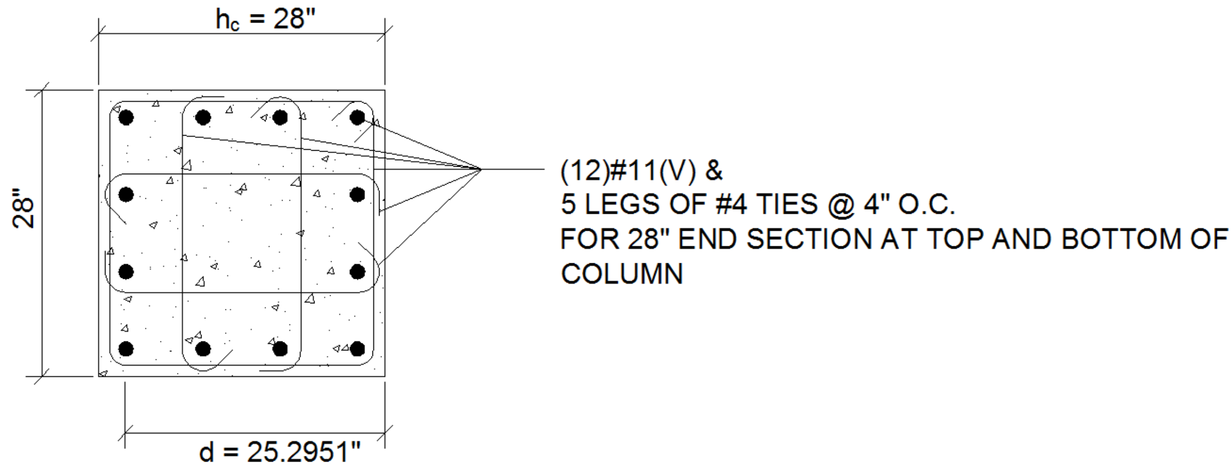

Figure B-38: Exterior column, cross-section at end section of column at ground floor level based on tsunami design requirements.

Center Section (B)

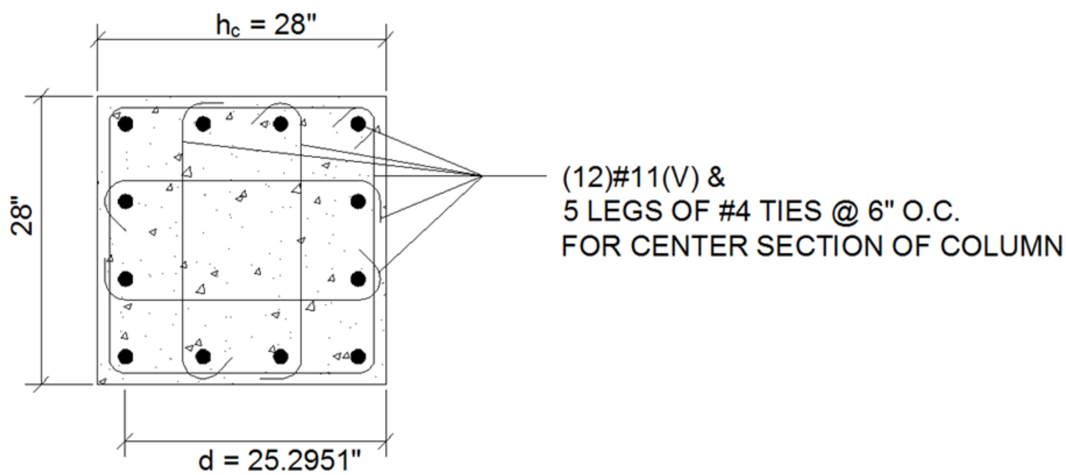

Figure B-39: Exterior column, cross-section at center section of column at ground floor level based on tsunami design requirements.

B.11.1.4 Exterior Column Shear Design

Critical Shears:

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 315.5 kips.

The shear capacities of the 28”x28” columns with 4 leg #5 Stirrups at 4” o.c. in the end section and 4 leg #5 Stirrups at 6” o.c. in the center section are given by:

$ØV_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4{,}500}\left(1 + \frac{315{,}500}{2{,}000\times28\times28}\right)28 \times 25.295/1{,}000$ = 114 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.31)\times60{,}000\times25.295}{4\times1{,}000}$ = 470 kips

$V_S = \frac{A_v f_y d}{s}$ = 470 kips $\not> 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4{,}000}\times28\times25.295 = 380$ kips $\therefore$ use 380 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.31)\times60{,}000\times25.295}{6\times1{,}000}$ = 314 kips

$V_S = \frac{A_v f_y d}{s}$ = 314 kips $\not> 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4{,}000}\times28\times25.295 = 380$ kips $\therefore$ use 314 kips

Therefore in the end sections, $ØV_n$ = 0.75 (114 + 380) = 371 k

and in the center sections, $ØV_n$ = 0.75 (114 + 314) = 321 k

At d: V_u = 365 k < ϕV_n = 371 k, therefore the column is adequate for shear at the edge.

At $d + h_c$*:* V_u = 265 k < ϕV_n = 321 k, therefore the column is adequate for shear at the center.

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = 315.5 kips.

The shear capacities of the 28"x28" columns with 3 leg #4 Stirrups at 4" o.c. in the end section and 3 leg #3 Stirrups at 6" o.c. in the center section are given by:

$ØV_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{289{,}250}{2{,}000\times28\times28}\right)28 \times 25.365/1{,}000$ = 106 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times25.365}{4\times1{,}000}$ = 228 kips

$V_S = \frac{A_v f_y d}{s}$ = 470 kips $\not> 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4{,}000}\times28\times25.365 = 359$ kips $\therefore$ use 228 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times25.365}{6\times1{,}000}$ = 84 kips

$V_S = \frac{A_v f_y d}{s}$ = 470 kips $\not> 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4{,}000}\times28\times25.365 = 359$ kips $\therefore$ use 84 kips

Therefore in the end sections, $ØV_n$ = 0.75 (106 + 228) = 251 k

and in the center sections, $ØV_n$ = 0.75 (106 + 84) = 143 k

At d: V_u = 69 k < ϕV_n = 251 k, therefore the column is adequate for shear at the edge.

At d + h_c*:* V_u = 97 k < ϕV_n = 143 k, therefore the column is adequate for shear at the center.

By inspection the remaining columns are adequate to resist the tsunami shear force.

Instead of the equivalent static load analysis performed above, it is permissible to use a non-linear analysis following the provisions of ASCE 41, or to perform a non-linear dynamic analysis of the column subjected to the debris impact strike.

B.11.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure B-15**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 24 in square column cross section shown in **Figure B-40** and **Figure B-41** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force for the end section of the column occurs at a distance "*d*" from the ends of the column, where *d* = 24 - 1.5 - 0.5 - 0.5 = 21.5 in. The critical shear force for the center section of the column occurs at "d + h_c" from the end of the column, where d + h_c = 21.5 + 24 = 45.5 in.

Floor 1 – 6

End Section (A)

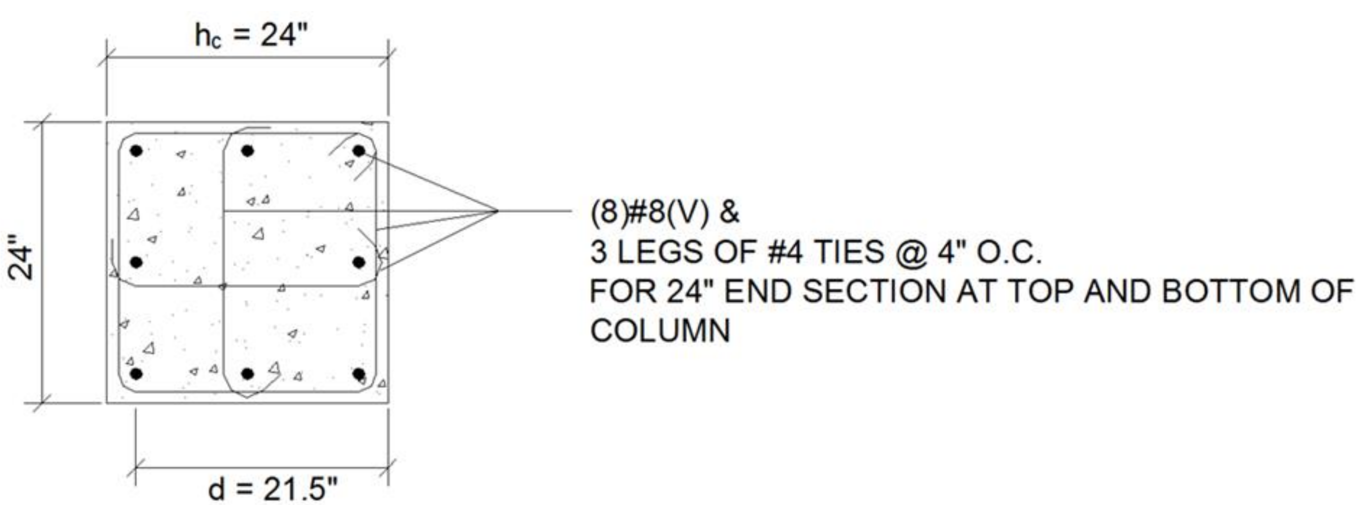

Figure B-40: Interior column, end section cross-section for column at all floor levels based on SDC D design.

Center Section (B)

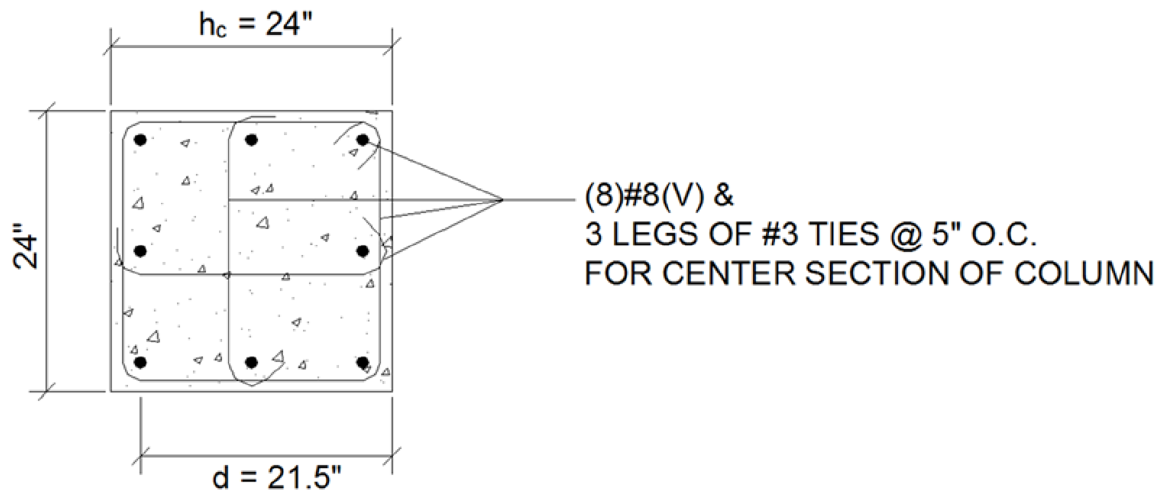

Figure B-41: Interior column, center section cross-section for column at all floor levels based on SDC D design.

B.11.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 29 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$$P_D = [120(28)(29)(6) + 2^2(150)(74)]/1000 = 629 \text{ k.}$$

Floor Live load reduction factor = 0.25+15/[4(29)(28)(5)]$^{0.5}$ = 0.367, therefore using 0.4 gives:

$$P_L = 0.4[95(5)+65(24)](28)(5)]/1000 = 114 \text{ k.}$$

Roof Live Load reduction factor = R_1R_2= 0.6(1.0) = 0.6 for A_t > 600 sf, therefore the roof live load is:

$$P_{Lr} = 0.6(20)(28)(29) = 9.7 \text{ k}$$

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

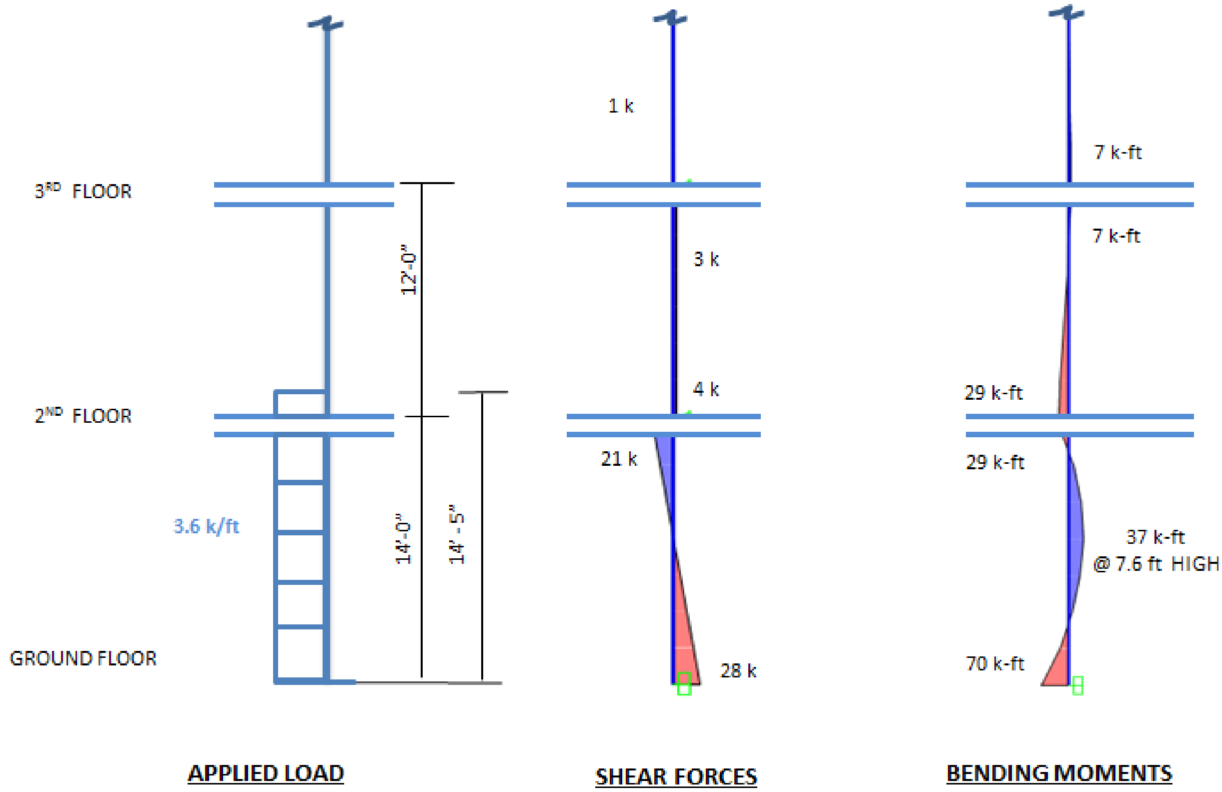

Figure B-42: Hydrodynamic loading on interior column of Hilo office building due to Load Case 2

Table B-5 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table B-5: Results from loading conditions of Monterey office building interior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
70	811.8	21	14	1.2D+Ftsu+0.5L (Hydro)
70	566.1	21	14	0.9D+Ftsu (Hydro)
Floor 2				
29	676.5	3	3	1.2D+Ftsu+0.5L (Hydro)
29	471.75	3	3	0.9D+Ftsu (Hydro)
Floor 3				
7	541.2	1	1	1.2D+Ftsu+0.5L (Hydro)
7	377.4	1	1	0.9D+Ftsu (Hydro)
Floor 4				
2	405.9	0	0	1.2D+Ftsu+0.5L (Hydro)
2	283.05	0	0	0.9D+Ftsu (Hydro)
Floor 5				
0	270.6	0	0	1.2D+Ftsu+0.5L (Hydro)
0	188.7	0	0	0.9D+Ftsu (Hydro)
Floor 6				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.35	0	0	0.9D+Ftsu (Hydro)

B.11.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure B-15** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure B-43 shows the interaction diagram for a typical interior column with the tsunami load combinations.

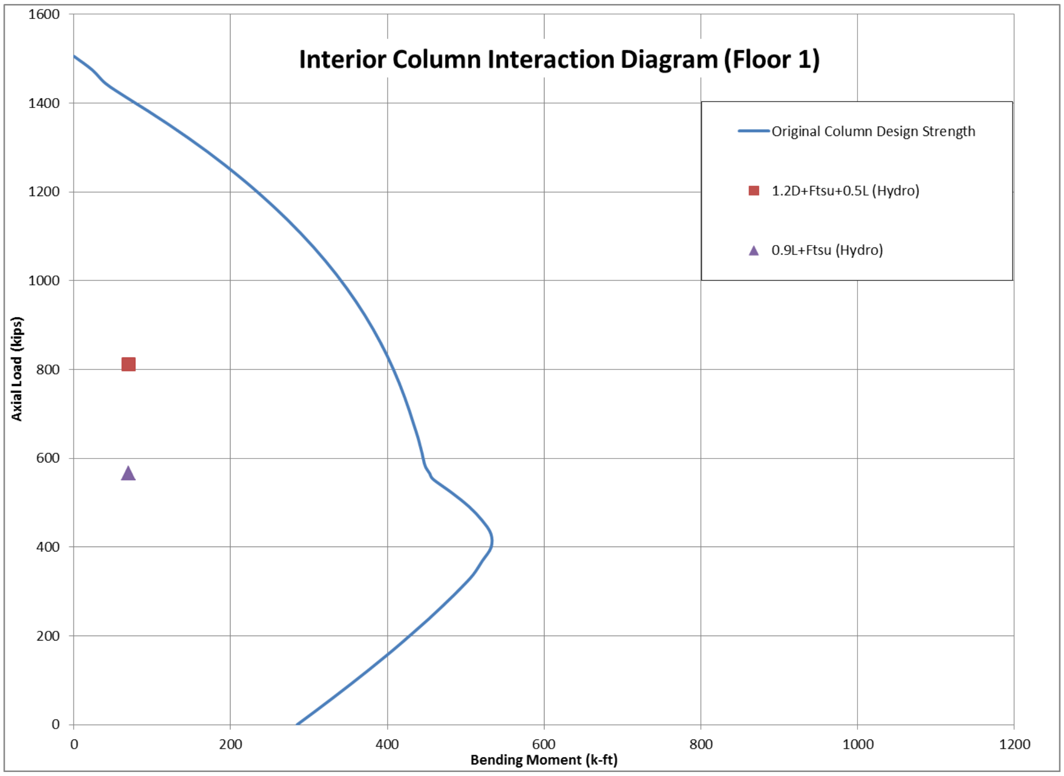

Figure B-43: Interaction diagram for typical ground floor office interior column showing tsunami load combinations

The existing interior column is therefore adequate at the first floor level, and by inspection the remaining columns are also adequate to resist the tsunami bending moments.

B.11.2.3 Interior Column Shear Design

Critical Shears:

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of ($1.2D + F_{TSU} + 0.5L$) per **Eqn. 6.8.3.3.-1a** P_u = 811.8 kips.

The shear capacities of the existing 24”x24” column with 3 leg #4 Stirrups @ 4” o.c. in the end sections and 3 leg #3 Stirrups @ 5” o.c. in the center section are given by:

$ØV_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{811{,}800}{2{,}000\times24\times24}\right)24\times21.5/1{,}000 = 111$ kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times21.5}{4\times1{,}000} = 194$ kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times21.5}{5\times1{,}000} = 85$ kips

Therefore in the end sections, $ØV_n = 0.75\ (111 + 194) = 229$ k

and in the center sections, $\emptyset V_n$ = 0.75 (111 + 85) = 147 k

At d: V_u = 43 k < ϕV_n = 229 k, therefore the column is adequate for shear at the edge

At d + h_c: V_u = 26 k < ϕV_n = 147 k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear force.

B.12 Tsunami Design for Residential Building

B.12.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

In lieu of performing detailed hydrostatic and hydrodynamic analysis, ***Eqn. 6.10.1-1*** *provides a simplified but conservative estimate of the maximum lateral load on the building.*

$$f_{uw} = 2.5 I_{tsu} \gamma_s h_{max}^2 = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 21.6^2 = 82.1\ kip/ft$$

Assuming $C_{cx} = 0.7$ *controls per* ***Section 6.8.7***

Therefore $F = 0.7 \times 254 \times 82.1 = 14{,}600\ kips$

This lateral load can be compared with 0.75 Ω_o E_h *= 0.75 x 2.5 x 2,259= 4,236 kips <14,600 kips. Therefore the LFRS is not adequate to satisfy this requirement and the detailed analysis for LC2 and LC3 shown below is recommended. The components can also be designed on the basis of this conservative uniform distributed force with the appropriate width b dimensions (but that is not illustrated here).*

B.12.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

B.12.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} $\leq h_{max}$ = 21.6 ft

$\leq$ 14 ft (ground floor story height)

$\leq$ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

Load Case 1 also requires application of the associated hydrodynamic drag on the entire building. However this will not control since buoyancy need not be considered.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2} \rho_s I_{tsu} C_d C_{cx} B (hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4575 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254' overall width of building

h = 8'

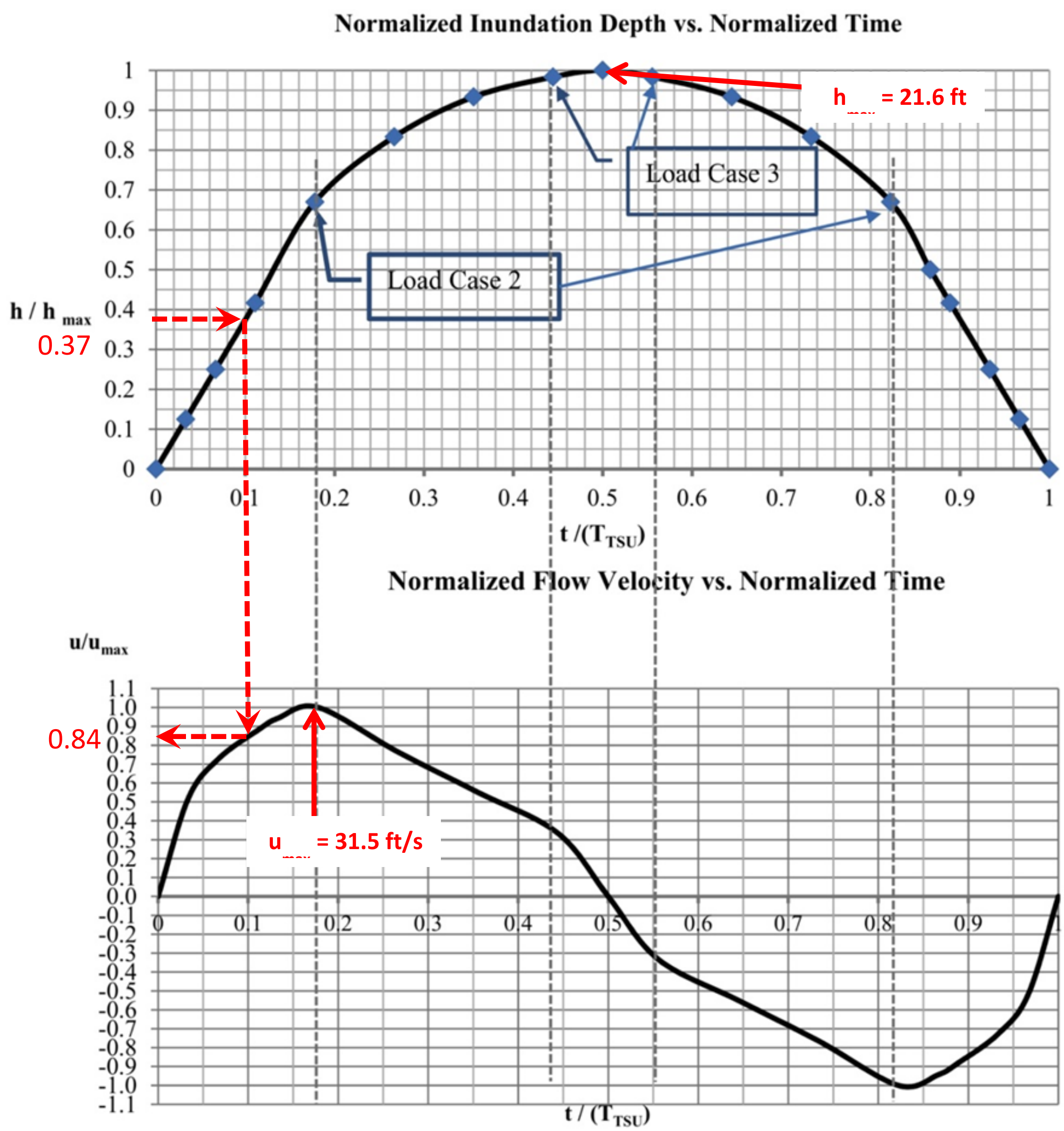

Figure B-44: Determining "u" for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/21.6 = 0.370. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.11. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.84. Therefore the flow velocity is u = 0.84 x 31.5 = 26.46 fps.

So$F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 26.46^2)/1000 = 2{,}281\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail to relieve lateral load on the structural frame. Note that a portion of this load will go to the ground floor slab, which reduces the load that has to be resisted by the lateral force resisting system. The entire lateral load must be resisted by the deep foundations assuming maximum scour has already occurred.

B.12.2.2 Load Case 2: Maximum Flow Velocity

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is 2/3h_{max} = 2/3 x 21.6 = 14.4 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.316 (**Table 6.10-1** based on B/h_{sx} = 254/14.4 = 17.6)

Since the inundation depth of 15.25 feet exceeds the bottom of the second floor slab (12' – 8"/12) = 11.33', the inundated area of the beams must be included in the closure coefficient, which is determined as follows:

h_{sx} = 14.4 ft.

A_{col} = 32 x 1.67' x (14.4'-0.67') = 734 ft^2

A_{wall} = 2 x 28' x (15.25'-0.67') + 2 x 10' x (15.25'-0.67') = 1109 ft^2

A_{Beam} = A_{Slab} = 1 x 254' x 0.67' + 1 x 254' x 0.67' = 169 ft^2

$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5\times A_{beam}}{Bh_{sx}} = \frac{\sum((734+1044)+1.5\times 169}{254\times 14.4} = 0.555 < 0.7$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = u_{max} = 31.5 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.316 \times 0.7 \times 254(14.4 \times 31.5^2)/1000 =$
$3{,}679\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 14.4 feet above grade. The lateral force resisting system for the structure must be evaluated for this load. During drawdown the same pressure needs to be applied to the inland elevation and the lateral force resisting system evaluated for this load.

B.12.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 21.6 ft. and the flow velocity is $1/3u_{max}$ = 1/3 x 31.5 = 10.5 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/21.6 = 11.75)

Since the inundation depth of 21.6 ft exceeds the third floor slab elevation of 21 ft, the closure coefficient is given by:

h_{sx} = 21.6 ft.

A_{col} = 32 x 1.67' x (21.6'-0.67'-0.67') = 1081 ft^2

A_{wall} = 2 x 28' x (21.6'-0.67'-0.67') + 2 x 10' x (21.6'-0.67'-0.67') = 1540 ft^2

$A_{Beam} = A_{Slab}$ = 2 x 254' x 0.67' = 339 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum(1081+1540)+1.5\times339}{254'\times21.6'} = 0.570 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 10.5 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(21.6 \times 10.5^2)/1000 = 582\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 21.6 feet above grade. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth.

B.12.2.4 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category D, **Section 6.8.3.4** permits the use of $0.75\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear. From the seismic design of this structure, E_h = 2,259 kips. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 2.5 \times 2{,}259 = 4{,}236\ kips$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 3,679 kips applied over a height of 14.4 ft. A portion of this load will be resisted by the grade beam/foundation system, reducing the overall load by 1,533 kips. (**Figure B-45**)

$$0.75\Omega_o E_h = 4{,}236\ kips > 2{,}146\ kips$$

Therefor the lateral force resisting system has ample capacity to resist the overall tsunami loads.

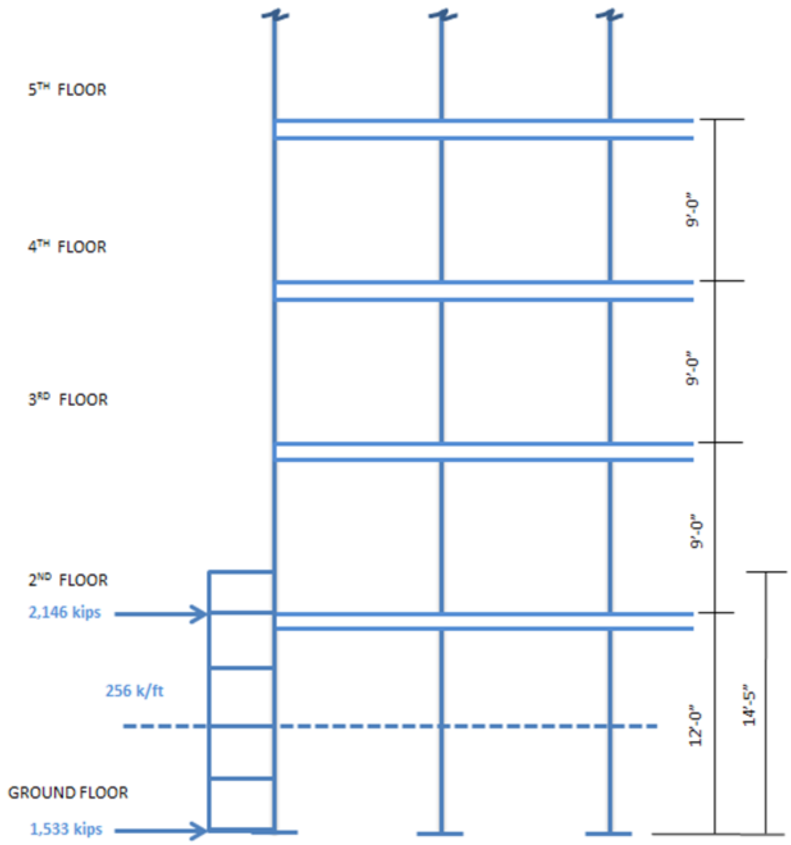

Figure B-45: LC2 Tsunami loads on overall Monterey Residential building

B.13 Component Design

B.13.1 Drag Force on Components - Section 6.10.2.2

B.13.1.1 Exterior Columns

Exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** requires that C_d = 2.0 and the width dimension, *b*, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Therefore *b* = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 14.4 ft and u_{max} = 31.5 fps.

The hydrodynamic drag is computed using **Eqn 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(14.4 \times 31.5^2)/1000 = 616\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 616/14.4 = 42.78 kips/ft over the lower 14.4 feet of the column. The column must be designed for this load combined with gravity loads per **Section 6.8.3.3**.

B.13.1.2 Interior Columns

Interior columns are 20" (1.67 ft) square R.C. columns. The controlling load case will be LC2, when the inundation depth is h_e = 14.4 ft and u_{max} = 31.5 fps.

The hydrodynamic drag is computed using **Eqn. 6.10-4** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**) and b = 1.67 ft since no debris accumulation is considered for interior column.

Therefore $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 1.67(14.4 \times 31.5^2)/1000 = 52.4\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 52.4/14.4 = 3.64 kips/ft over the lower 14.4 feet of the column. This load must be combined with gravity loads per **Section 6.8.3.3** and the column capacity verified.

B.13.2 Tsunami Loads on Structural Walls, F_w – Section 6.10.2.3

Since tsunami bores are anticipated at this location, the lateral load on the structural walls is given by **Eqn. 6.10-5a** or **Eqn. 6.10-5b**, depending on the flow depth relative to the wall width:

Eqn. 6.10-5a $F_W = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$

Eqn. 6.10-5b $F_W = \frac{3}{4}\rho_s I_{tsu} C_d b(h_e u^2)$ when $\frac{b_w}{h_e} \geq 3$

Where C_d = 2.0 for a wall per **Table 6.10-2**, and

Elevator Walls:

b = 28' for the elevator walls

Elevator $\frac{b_w}{h_e} = \frac{28'}{14.4'} = 1.94 \ngeq 3 \therefore$ Eqn. 6.10-5a applies.

The controlling load case will be LC2, where h_e = 14.4 ft and u = 31.5 fps.

Therefore, for the 28' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 28(14.4 \times 31.5^2)/1000 = 880\ kips$

This load is applied to the wall as a uniformly distributed pressure of 880/(28x14.4) = 2,183 psf over the lower 14.4 ft of the wall.

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3} \rightarrow h_e \geq \frac{b_w}{3} = \frac{28'}{3}$ = 9.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 9.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure B-46**. The inundation ratio is h/h_{max} = 9.33'/21.6' = 0.432. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.12. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.9. Therefore the flow velocity is u = 0.9 x 31.5 = 28.4 fps. The bore loading is computed using **Eqn. 6.10-5b** for h_e = 9.33 ft and u = 28.4 fps.

Therefore for the 28' wide elevator wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 28(9.33 \times 28.4^2)/1000 = 695\ kips$

This load is applied to the wall as a uniformly distributed pressure of 695/(28x9.33) = 2,660 psf over the lower 9.33 ft of the wall.

The bending moments and shears in the wall must be checked for both of these loading conditions.

Stairwell Walls:

b = 10' for the elevator walls

Stairwell $\frac{b_w}{h_e} = \frac{10'}{14.4'} = 0.69 \ngeq 3 \therefore \text{Eqn. 6.10-5a}$

The controlling load case will be LC2, where h_e = 14.4 ft and u = 34 fps.

Therefore, for the 10' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 10(14.4 \times 31.5^2)/1000 = 314\ kips$

This load is applied to the wall as a uniformly distributed pressure of 314/(10x14.4) = 2,183 psf over the lower 14.4 ft of the wall. As expected, this pressure is the same as for the 28' wide elevator wall using the same Eqn. 6.10-5a.

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3} \rightarrow h_e \geq \frac{b_w}{3} = \frac{10'}{3}$ = 3.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 3.33 ft, the flow velocity can be obtained from **ASCE 7-16 Figure 6.8-1**, as shown in **Figure B-46**. The inundation ratio is h/h_{max} = 3.33'/21.6 = 0.154.

Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.04. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.6. Therefore the flow velocity is u = 0.6 x 31.5 = 18.9 fps. The bore loading is computed using **Eqn. 6.10-5b** for h_e = 3.33 ft and u = 18.9 fps.

Therefore for the 10' wide stairwell wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 10(3.33 \times 18.9^2)/1000 = 39.3\ kips$

This load is applied to the wall as a uniformly distributed pressure of 39.3/(10x3.33) = 1,180 psf over the lower 3.33 ft of the wall. This will not govern when compared with the pressure due to hydrodynamic drag from Eqn. **6.10-5a.**

The bending moments and shears in the wall must be checked for a pressure of 2183 psf applied over the lower 14.4 ft of the wall.

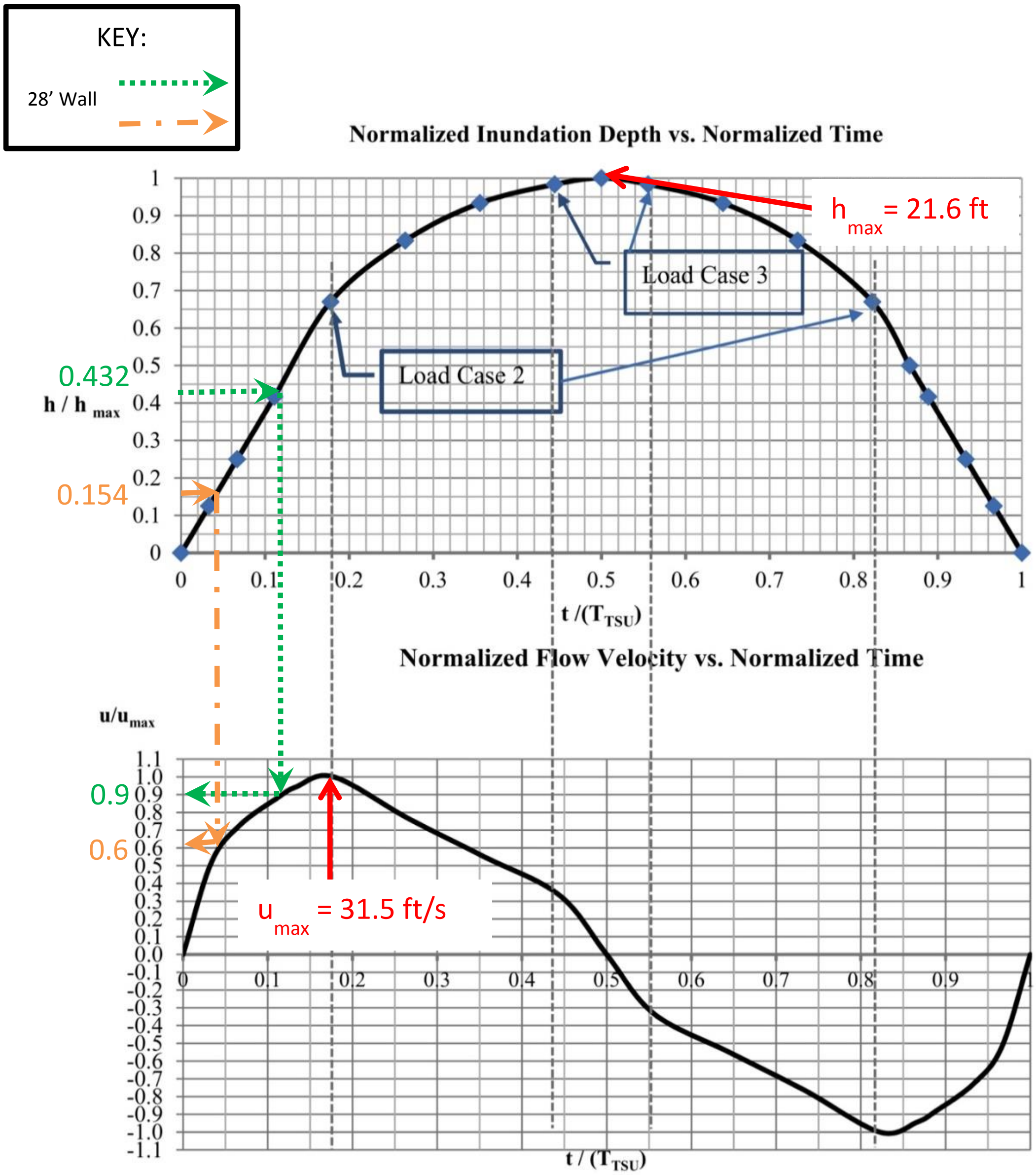

Figure B-46: Determining "u" for Eqn. 6.10-5b with ASCE 7-16 Figure 6.8-1

B.13.3 Hydrodynamic Pressures associated with Slabs – Section 6.10.3

B.13.3.1 Flow Stagnation Pressure – Section 6.10.3.1

The Mechanical/Electrical room on Gridline D between Gridlines 5 and 6 is enclosed on all sides by structural walls. Tsunami flow entering through the two door openings will result in flow stagnation pressurization of this room, given by **Eqn. 6.10-8** as:

$$P_p = \frac{1}{2}\rho_s I_{tsu} u^2$$

Assuming that the door openings are 7 ft high, the stagnation pressurization is based on the maximum flow velocity occurring at this or greater depths, i.e. when the door opening is fully submerged. The flow

velocity will therefore be the maximum of 31.5 fps which occurs when the flow depth is 14.4 ft (**Figure 6.8-1**, LC2). Therefore;

$$P_p = \frac{1}{2} \times 2.2 \times 1.0 \times 31.5^2 = 1091\, psf$$

The structural walls surrounding this room must be evaluated for an outward pressure of 356 psf, in combination with gravity loads per **Section 6.8.3.3**. The floor slab above this room must be designed for a net uplift pressure given by $0.9D + F_{TSU}$ = -0.9x100 + 1,091 = 1,001 psf upwards. This will require additional top reinforcement in this slab and shear reinforcement around the slab perimeter. In order to reduce the amount of additional reinforcement, one could perform a non-linear analysis of the floor slab following the provisions of ASCE 41. A simpler alternative may be to design the floor slab in the mechanical room as a breakaway slab, as shown in **Figure B-47,** in order to relieve pressure. This will apply to all levels up to h_{max}

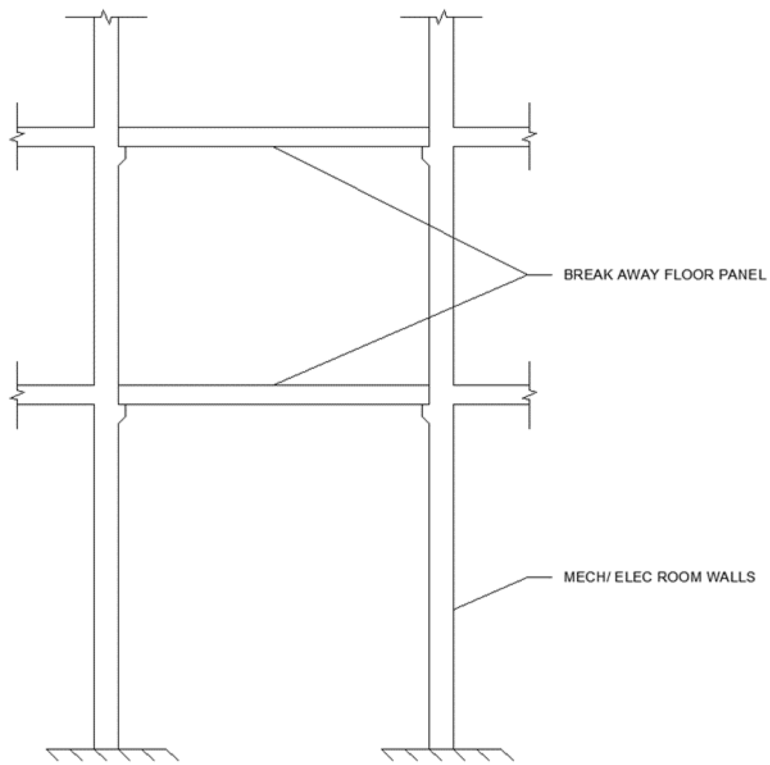

Figure B-47: Mechanical/Electrical room break-away floor panels applied to all levels up to h_{max}

B.13.3.2 Hydrodynamic Surge Uplift at Horizontal Slabs – Section 6.10.3.2

If slabs are submerged during the tsunami, they must be designed for uplift, with a specified minimum of 20 psf (**Section 6.10.3.2.1**). The uplift may increase if the ground floor is sloped, causing an upward component of flow velocity (**Section 6.10.3.2.2**). This is not the case for this building.

The resulting minimum uplift of 20 psf is much smaller than the dead weight of the slab (100 psf), therefore this uplift will not affect the slab design.

B.13.3.3 Tsunami Bore Flow Entrapped in Structural Wall-Slab Recesses – Section 6.10.3.3

If a tsunami bore is entrapped in a structural wall-slab recess, then large pressures can develop on the slab and wall (**Section 6.10.3.3.1**). Although tsunami bores are anticipated at this location, the flow can pass freely around the wall elements in this building. Therefor this condition does not apply.

B.14 Debris Impact Loads - Section 6.11

The inundation depth exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads.

B.14.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330 I_{tsu} C_o = 330 \times 1.0 \times .65 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force can be reduced to 50%, or 107.25 kips. This load must be applied to the 20" square exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with other tsunami loads and it need not be applied to interior columns.

This equivalent static impact load of 107.25 kips must also be applied to any structural walls on the perimeter of the building. This applies to the 28 ft wide elevator walls on both exterior sides of the building (GLs A and D) since impact must be considered during inflow and outflow conditions. Evaluation of the wall capacity is based on a tributary wall width of half the wall height. Since the wall unbraced height is (12' – 8"/12) = 11.33', the tributary width is 5.67 ft.

B.15 Column Design for Tsunami Loads

B.15.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 from **Figure B-16**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure B-48** and **Figure B-49** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the end of the column, where *d* = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "d + h_e" from the end of the column, where d + h_e = 17.5 + 20 = 37.5 in.

Floor 1 – 7

End Section (A)

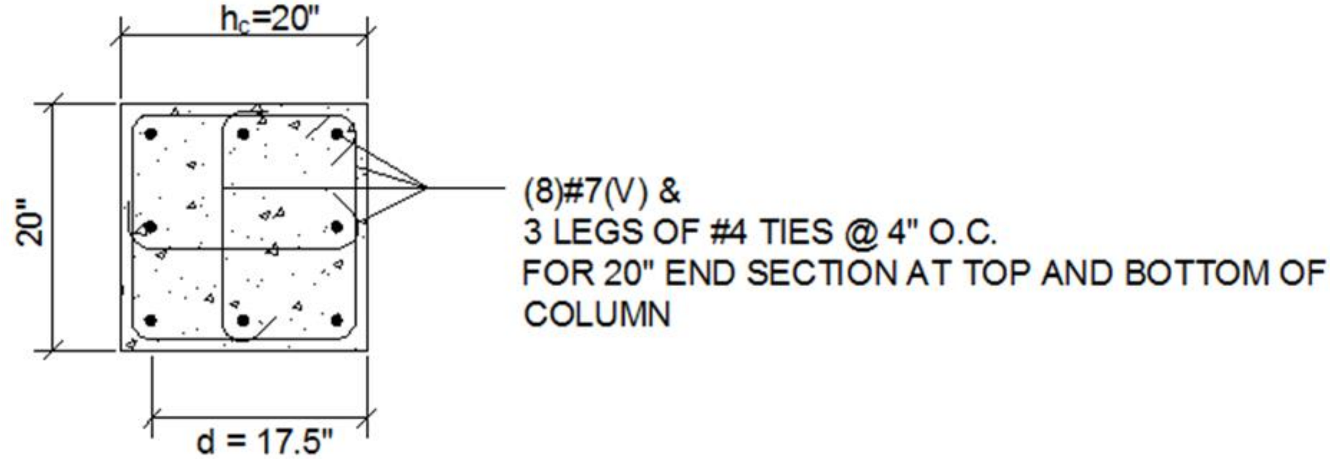

Figure B-48: Exterior column, cross-section at end of column at all floor levels based on SDC D design.

Center Section (B)

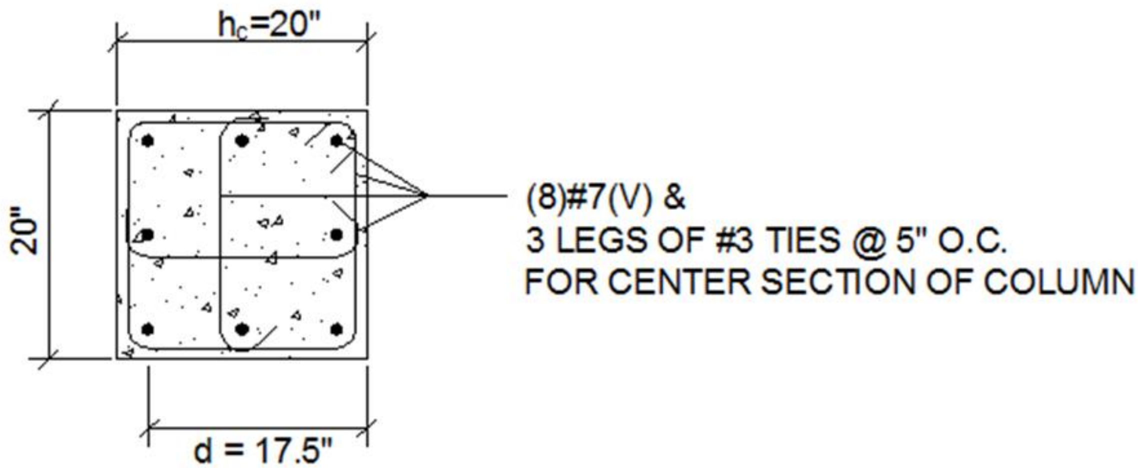

Figure B-49: Exterior column, cross-section at center of column at all floor levels based on SDC D design.

B.15.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 14.58 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

$$P_D = [128(14.58)(28)(6)+ 123(14.58)(28)+ 90(28)(6)+1.67^2(150)(66)]/1000 = 406 \text{ k}$$

Floor Live load reduction factor = $0.25+15/[4(14.58)(28)(5)]^{0.5} = 0.402$*, therefore, column base live load is:*

$$P_L = 0.402[55(14.58)(28)(6)]/1000 = 54.2 \text{ k}$$

Roof Live Load reduction factor = $R_1R_2= [1.2-(0.001)(14.58)(28)](1.0) = 0.792$*, therefore, roof live load is:*

$P_{Lr} = 0.792(20)(14.58)(28)/1000 = 6.47kF$

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

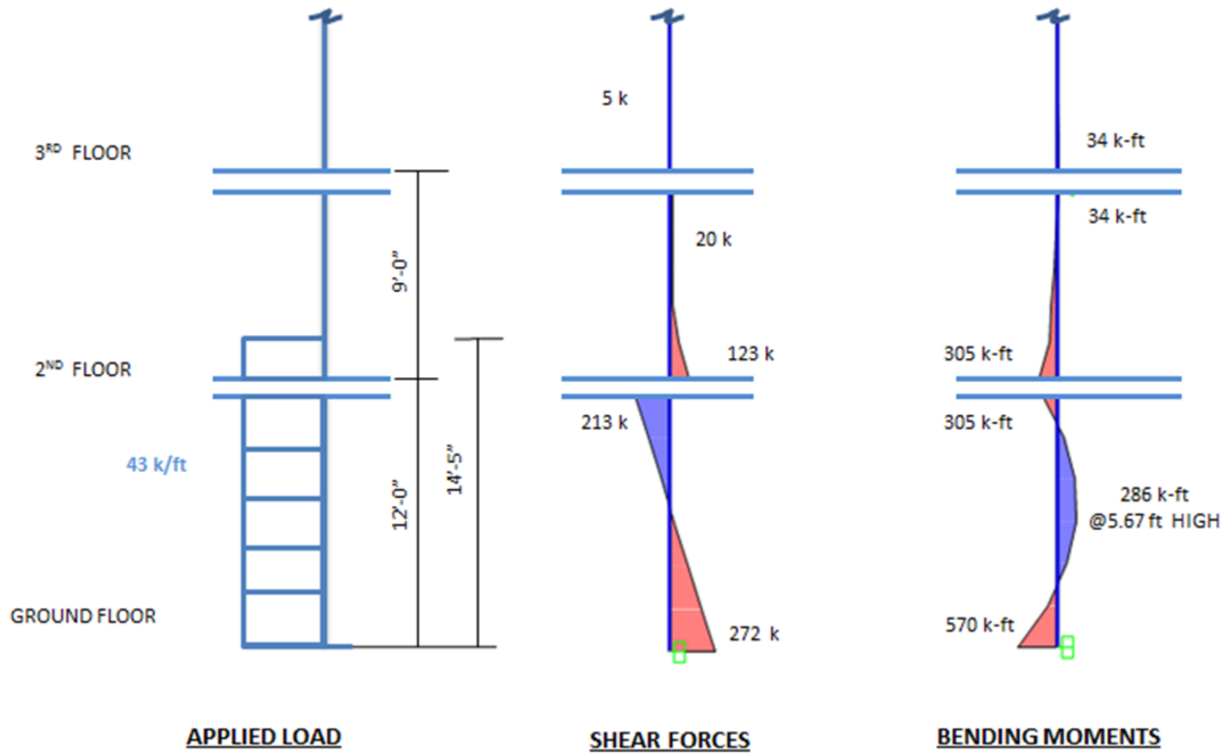

Figure B-50: Hydrodynamic loading on exterior column of Monterey residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just above and below each inundated floor slab for the maximum shear and near the mid-height of the clear column height for maximum bending moments. Samples of the resulting shear force and bending moment diagrams are provided below. Similar diagrams and similar shear and bending moments would result if the impact load was applied at the other end of each column.

Impact load at d:

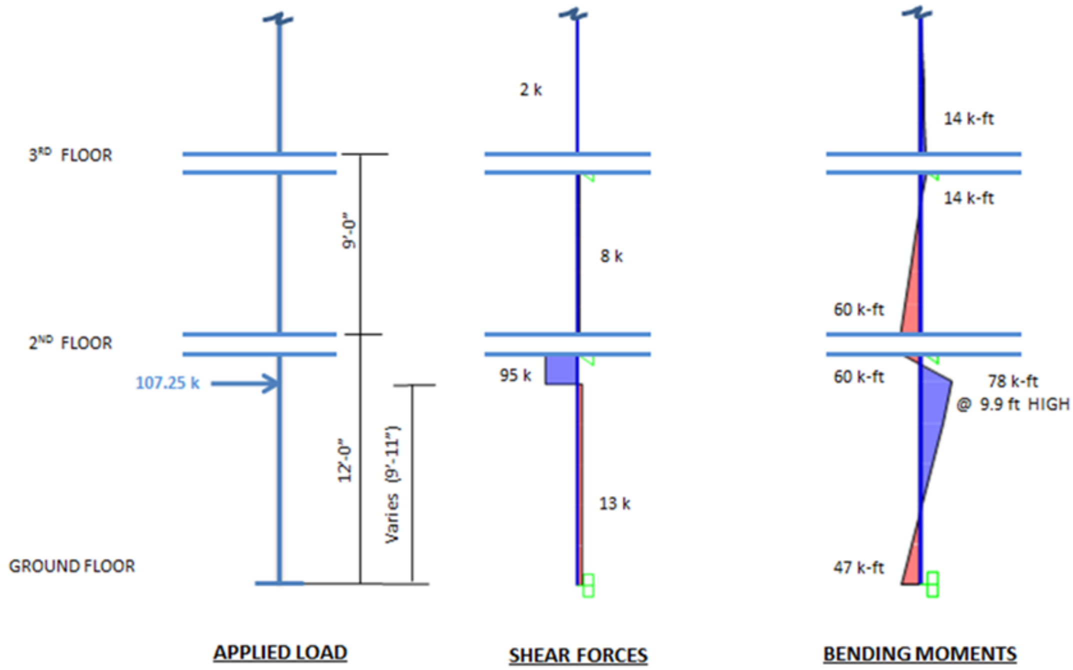

Figure B-51: Impact load applied at "d" away from the end of column on the ground floor

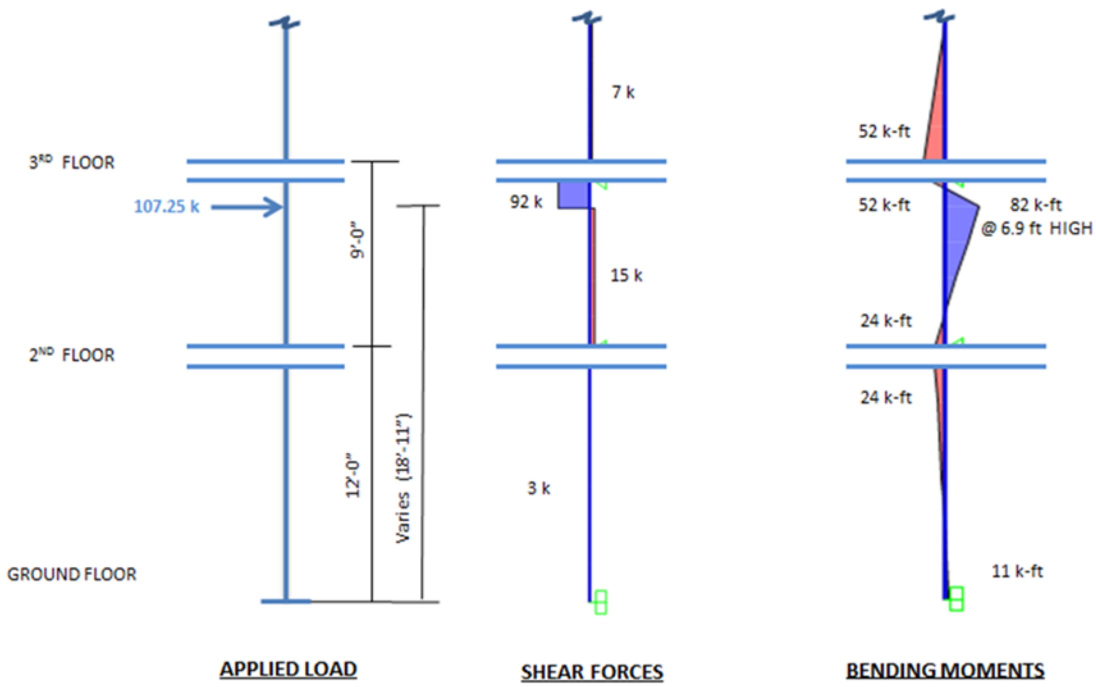

Figure B-52: Impact load applied at "d" away from the end of column on the 2nd floor

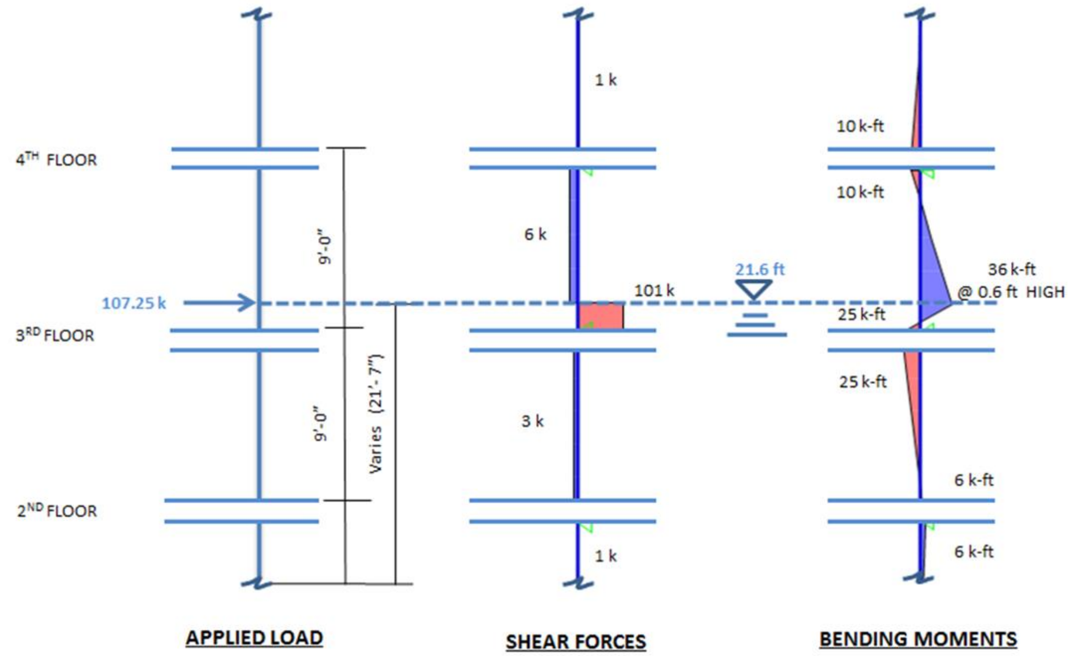

Figure B-53: Impact load applied at 0.6 ft away instead of "d" as water level is lower than "d" away from the end of column on the 3rd floor

Impact load at d + h_c:

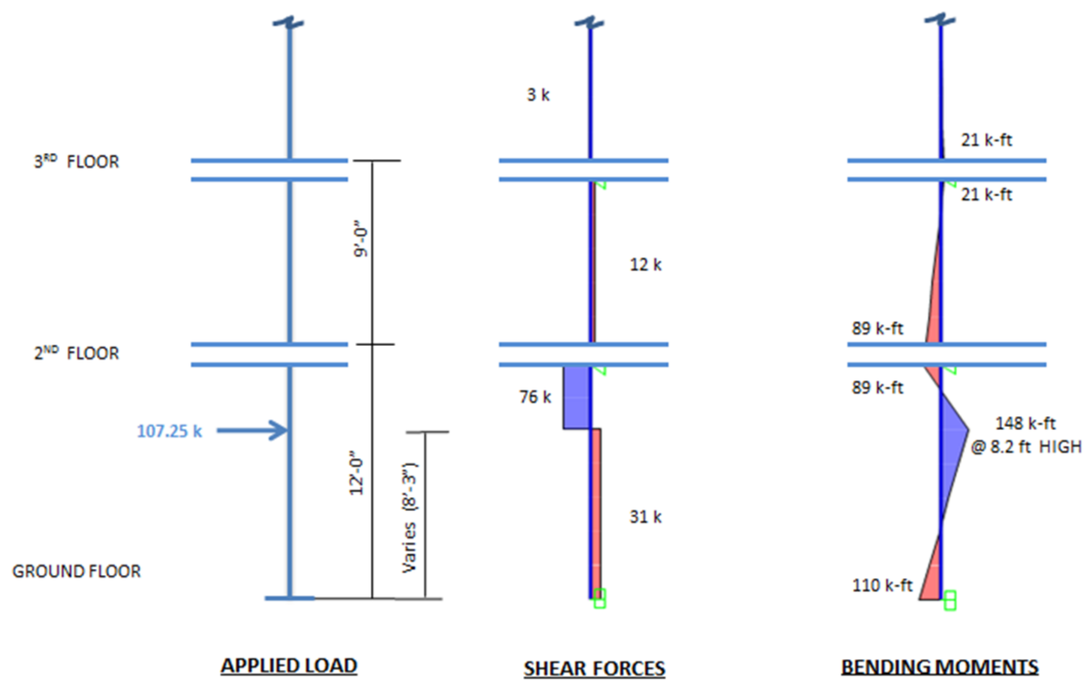

Figure B-54: Impact load applied at "d + h_c" away from the end of column on the ground floor

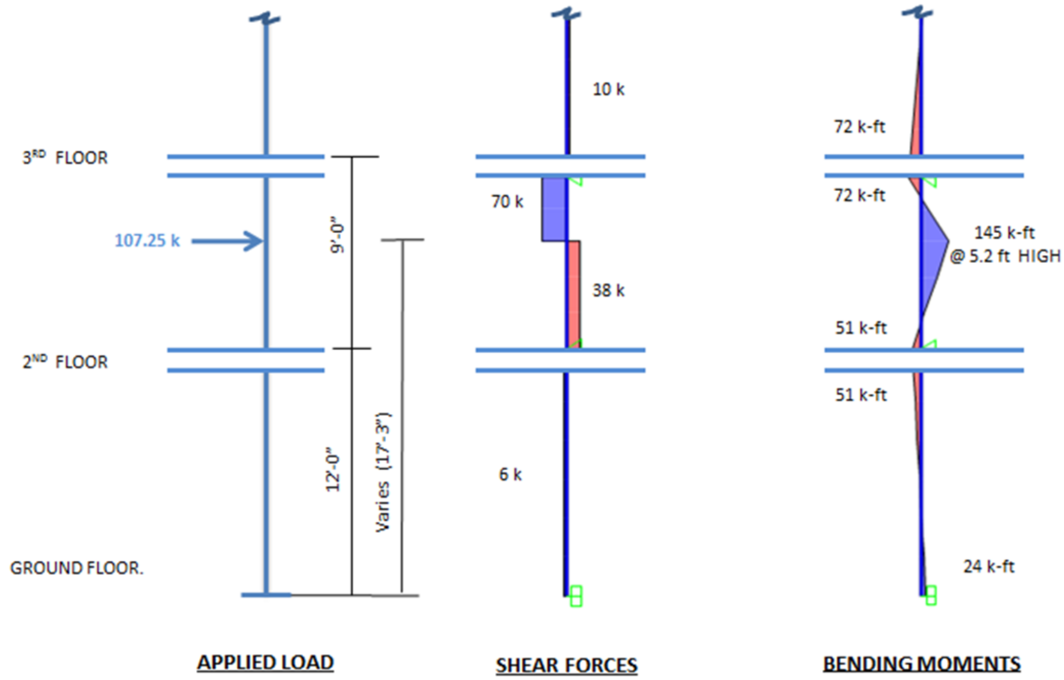

Figure B-55: Impact load applied at "d + h_c" away from the end of column on the 2nd floor

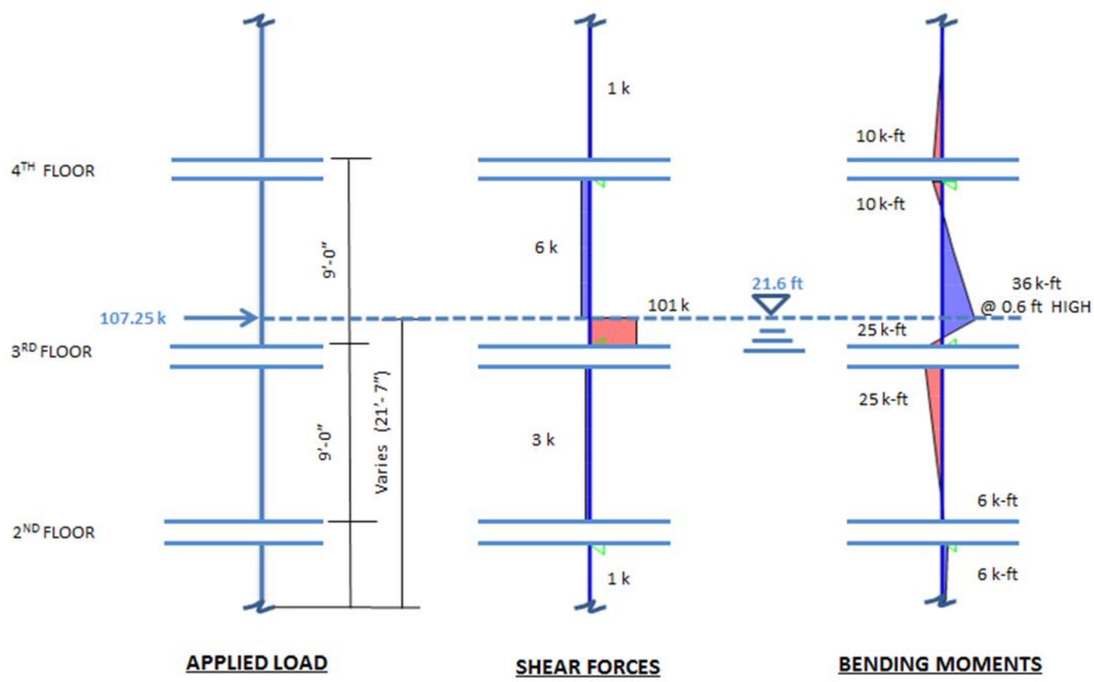

Figure B-56: Impact load applied at 0.6 ft away instead of "d + h_c" as water level is lower than "d + h_c" away from the end of column on the 3rd floor

Impact load at mid-height:

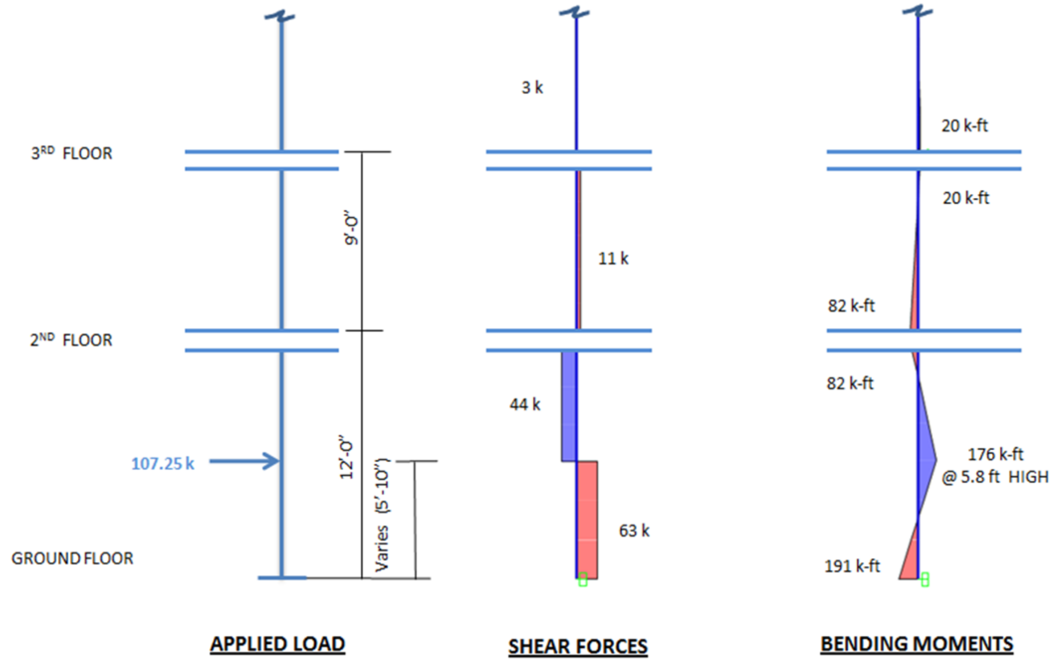

Figure B-57: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

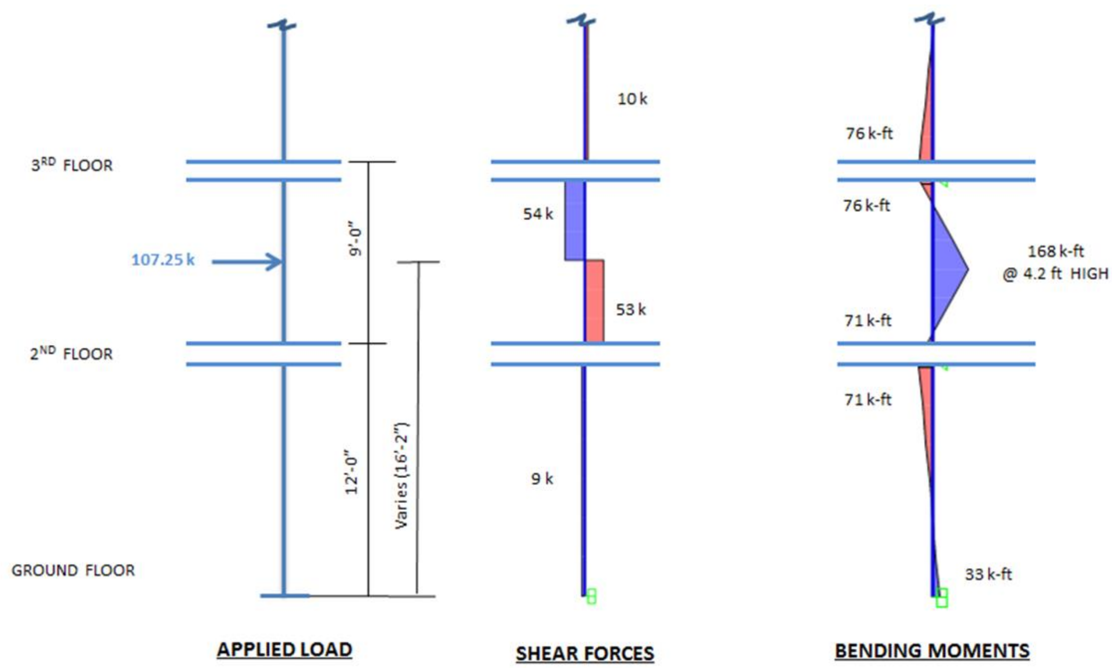

Figure B-58: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

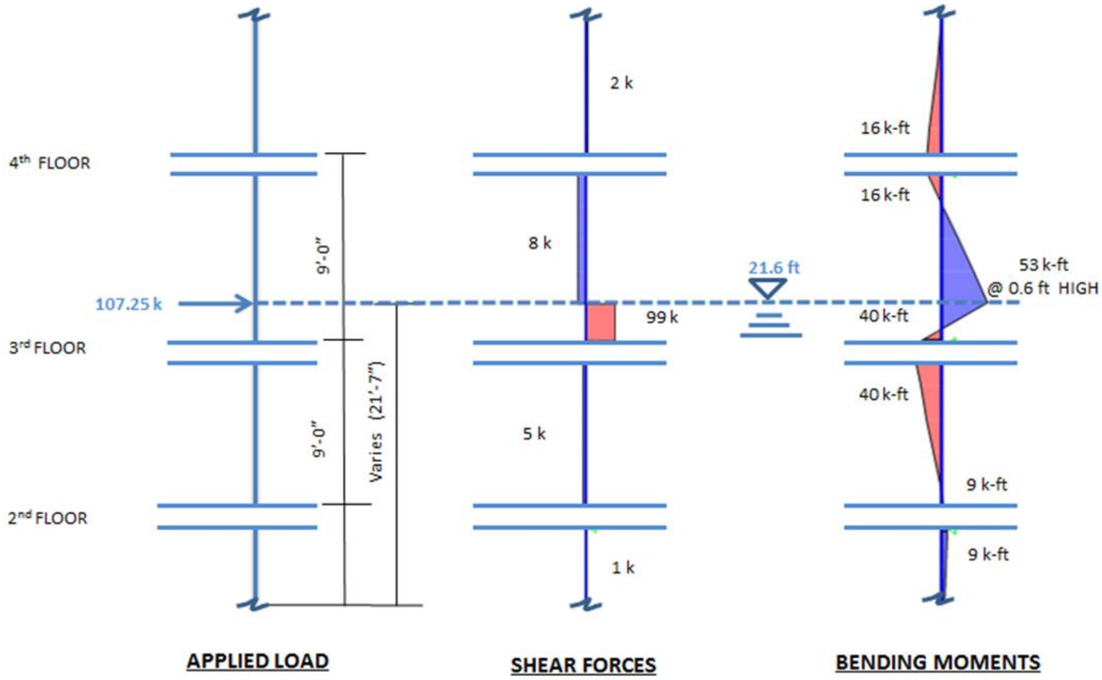

Figure B-59: Impact load applied at 0.6 ft away instead of mid-height as water level is lower than mid-height away from the end of column on the 3rd floor

Table B-6 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and long impact (Impact). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table B-6 : Results from loading conditions of Monterey residential building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
570	514.3	210	139	1.2D+Ftsu+0.5L (Hydro)
570	365.4	210	139	0.9D+Ftsu (Hydro)
191	514.3	95	76	1.2D+Ftsu+0.5L (Impact)
191	365.4	95	76	0.9D+Ftsu (Impact)
Floor 2				
305	440.8	60	24	1.2D+Ftsu+0.5L (Hydro)
305	313.2	60	24	0.9D+Ftsu (Hydro)
168	440.8	92	70	1.2D+Ftsu+0.5L (Impact)
168	313.2	92	70	0.9D+Ftsu (Impact)
Floor 3				
34	367.4	5	5	1.2D+Ftsu+0.5L (Hydro)
34	261	5	5	0.9D+Ftsu (Hydro)
53	367.4	101	6	1.2D+Ftsu+0.5L (Impact)
53	261	101	6	0.9D+Ftsu (Impact)
Floor 4				
8	293.9	1	1	1.2D+Ftsu+0.5L (Hydro)
8	208.8	1	1	0.9D+Ftsu (Hydro)
16	293.9	1	1	1.2D+Ftsu+0.5L (Impact)
16	208.8	1	1	0.9D+Ftsu (Impact)
Floor 5				
2	220.4	0	0	1.2D+Ftsu+0.5L (Hydro)
2	156.6	0	0	0.9D+Ftsu (Hydro)
4	220.4	0	0	1.2D+Ftsu+0.5L (Impact)
4	156.6	0	0	0.9D+Ftsu (Impact)
Floor 6				
0	146.9	0	0	1.2D+Ftsu+0.5L (Hydro)
0	104.4	0	0	0.9D+Ftsu (Hydro)
1	146.9	0	0	1.2D+Ftsu+0.5L (Impact)
1	104.4	0	0	0.9D+Ftsu (Impact)
Floor 7				
0	73.5	0	0	1.2D+Ftsu+0.5L (Hydro)
0	52.2	0	0	0.9D+Ftsu (Hydro)
0	73.5	0	0	1.2D+Ftsu+0.5L (Impact)
0	52.2	0	0	0.9D+Ftsu (Impact)

B.15.1.2 Combined Gravity and Tsunami Loads

The column chosen at Grid Intersection A-3 from **Figure B-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure B-60 to **Figure B-62** shows the interaction diagram for the typical exterior column including the tsunami load combinations.

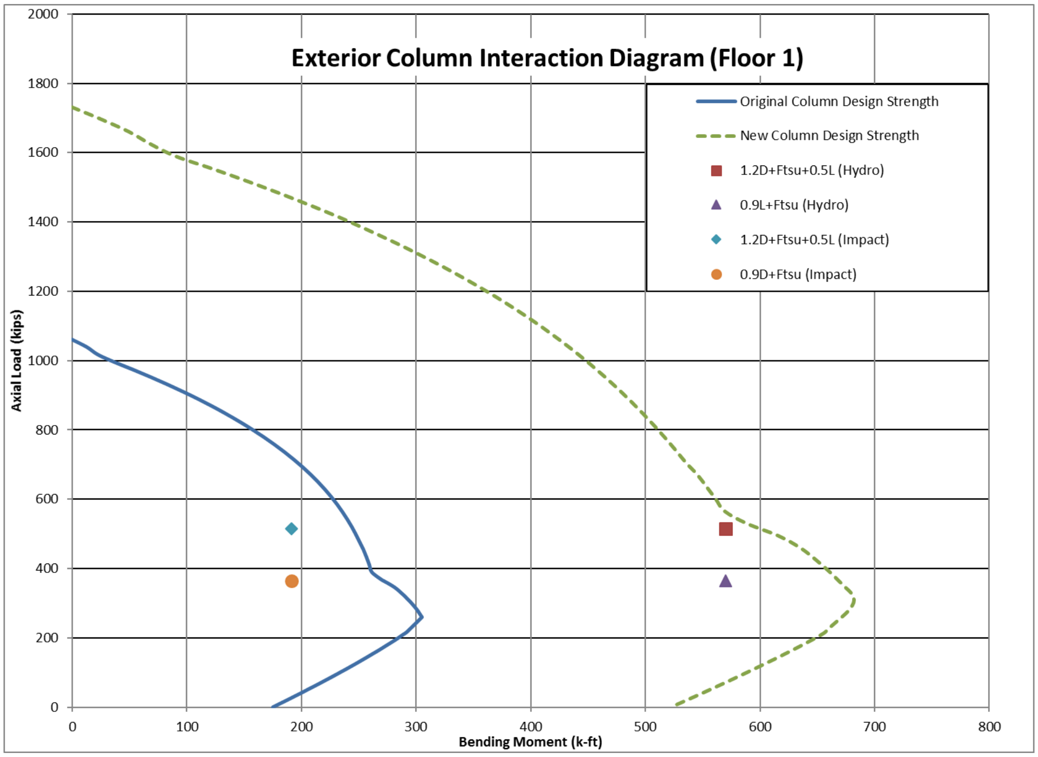

Figure B-60: Interaction diagram for typical ground floor exterior column showing tsunami load combinations

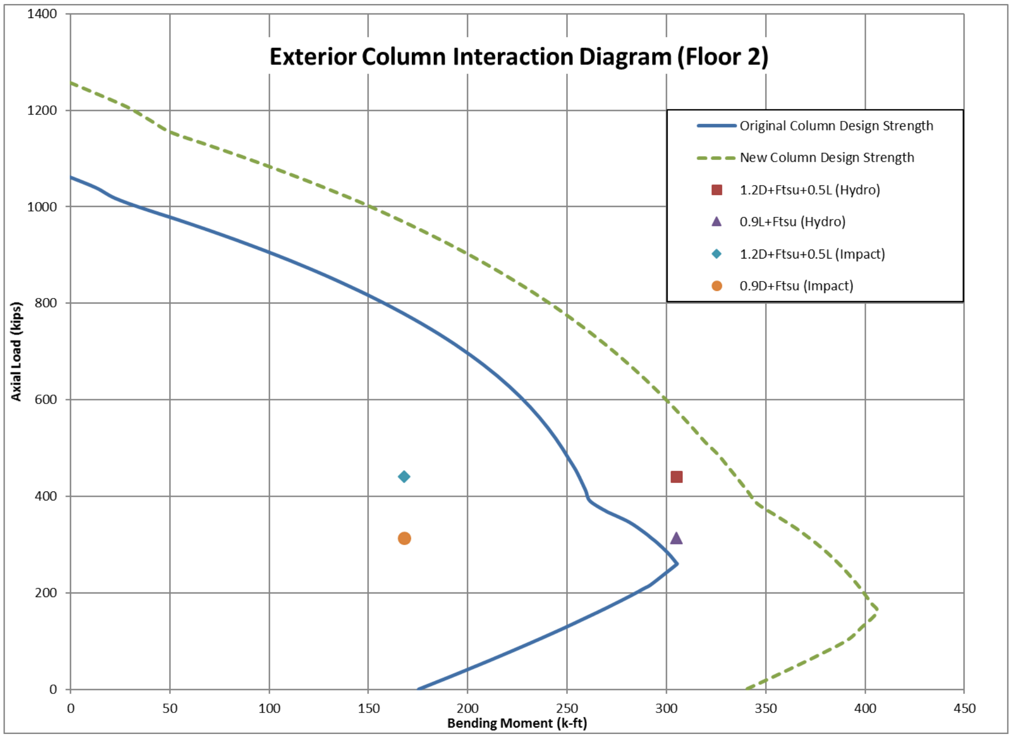

Figure B-61: Interaction diagram for typical 2nd floor exterior column showing tsunami load combinations

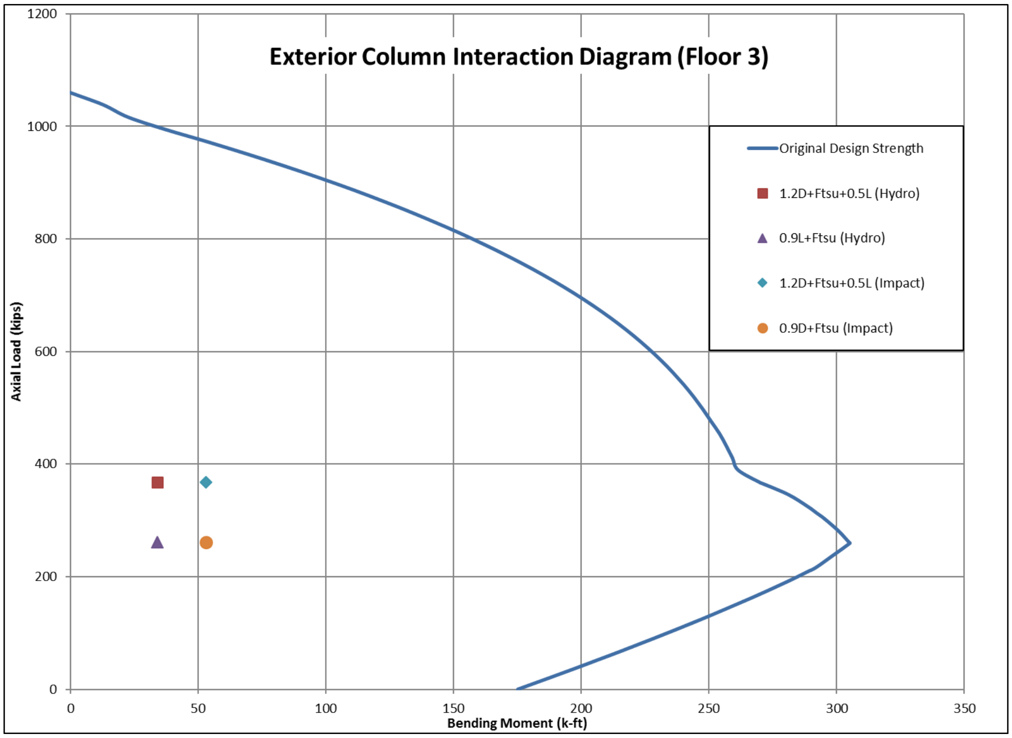

Figure B-62: Interaction diagram for typical 3rd floor exterior column showing tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

B.15.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure B-60** to **Figure B-62** the original exterior columns are adequate for log impact load, but the columns at the ground and 2nd floors must be strengthened to resist bending due to the hydrodynamic loads. Revised columns designs were developed to satisfy the hydrodynamic loads as shown in in **Figure B-63** to **Figure B-66**. The interaction diagrams for these new columns are shown in **Figure B-60** to **Figure B-61**.

Floor 1

End Section (A)

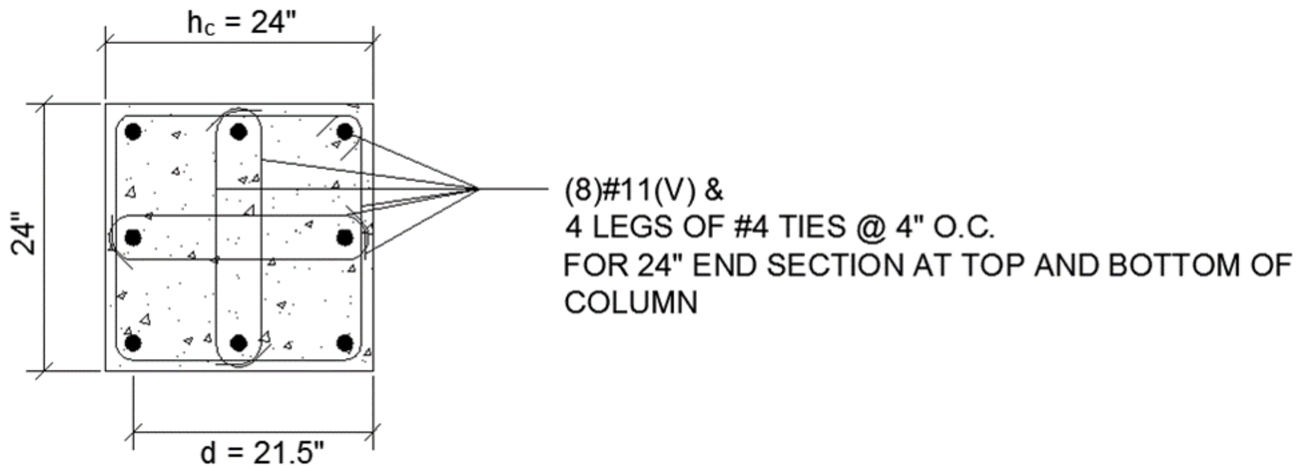

Figure B-63: Exterior column, cross-section at end of column at ground floor level based on tsunami design requirements.

Center Section (B)

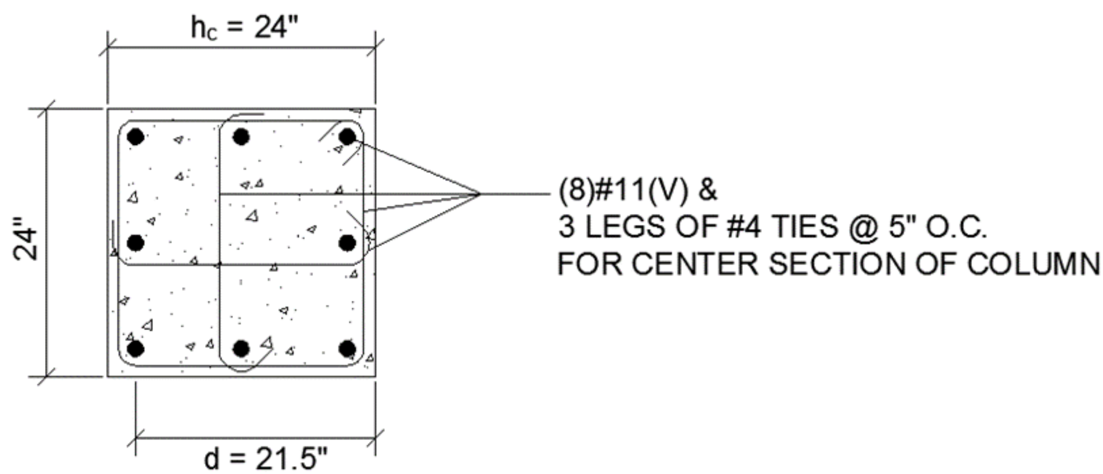

Figure B-64: Exterior column, cross-section at center of column at ground floor level based on tsunami design requirements.

Floor 2

End Section (A)

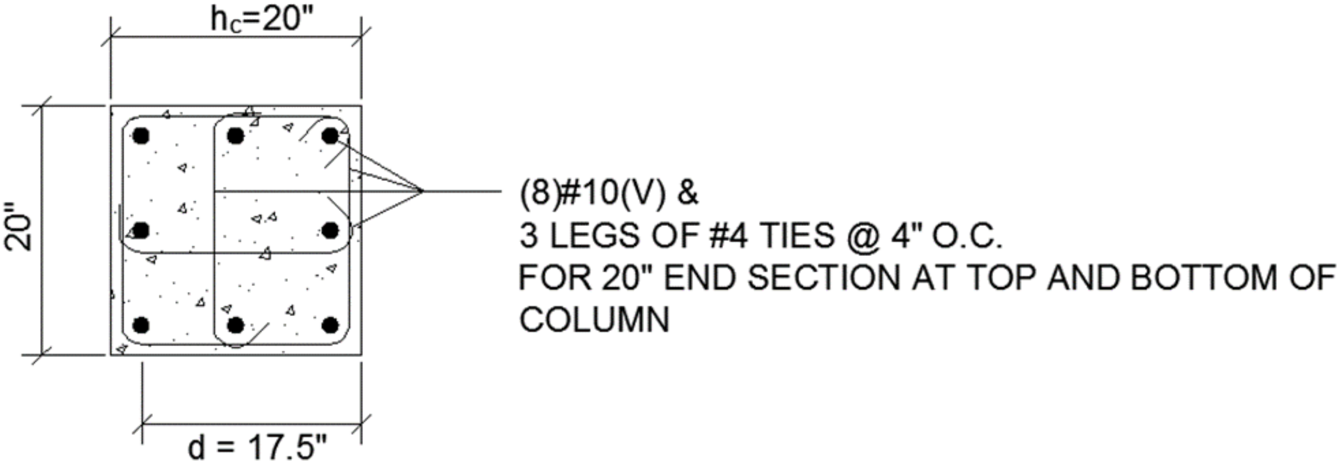

Figure B-65: Exterior column, cross section at end of column at the 2nd floor level based on tsunami design requirements.

Center Section (B)

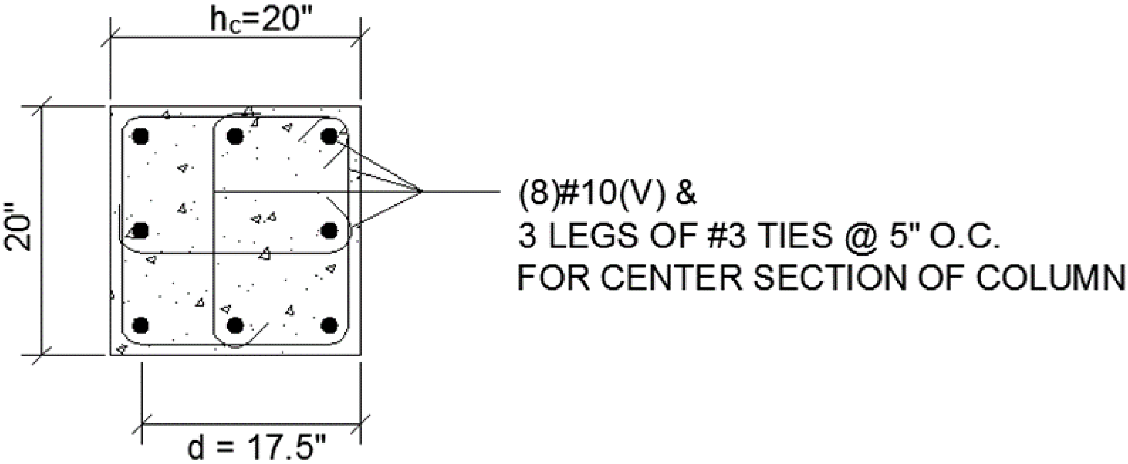

Figure B-66: Exterior column, cross-section at center of column at the 2nd floor level based on tsunami design requirements.

B.15.1.4 Exterior Column Shear Design

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = 365.4 kips.

The shear capacities of the 24”x24” columns with 4 leg #4 Stirrups at 4” o.c. in the end section and 3 leg #4 Stirrups at 5” o.c. in the center section are given by:

$$\emptyset V_n = \emptyset(V_C + V_S)$$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{365,400}{2,000\times 24\times 24}\right)24\times 21.295/1,000$ = 85 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times 0.2)\times 60,000\times 21.295}{4\times 1,000}$ = 256 kips

$V_S = \frac{A_v f_y d}{s}$ = 256 kips $\ngtr 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4,000}\times 24\times 21.295 = 259\text{ kips} \therefore \text{use } 256\text{ kips}$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.2)\times 60,000\times 21.295}{5\times 1,000}$ = 153 kips.

$V_S = \frac{A_v f_y d}{s}$ = 153 kips $\ngtr 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4,000}\times 24\times 21.295 = 259\text{ kips} \therefore \text{use } 153\text{ kips}$

Therefore in the end sections, $ØV_n$ = 0.75 (85 + 256) = 256 k

and in the center sections, $ØV_n$ = 0.75 (85 + 153) = 179 k.

At d: V_u = 210 k < ϕV_n = 256 k, therefore the column is adequate for shear at the end sections.

At d + h_c*:* V_u = 139 k < ϕV_n = 179 k, therefore the column is adequate for shear at the center section

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 313.2 kips.

The shear capacities of the 20"x20" columns with 3 leg #4 Stirrups at 4" o.c. in the end section and 3 leg #3 Stirrups at 5" o.c. in the center section are given by:

$ØV_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{313,200}{2,000\times 20\times 20}\right)20\times 17.365/1,000$ = 61 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.2)\times 60,000\times 17.365}{4\times 1,000}$ = 156 kips

$V_S = \frac{A_v f_y d}{s}$ = 156 kips $\ngtr 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4,000}\times 24\times 21.295 = 259\text{ kips} \therefore \text{use } 156\text{ kips}$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.11)\times 60,000\times 17.365}{5\times 1,000}$ = 69 kips.

$V_S = \frac{A_v f_y d}{s}$ = 69 kips $\ngtr 8\sqrt{f_C'}\text{ b d} = 8\times\sqrt{4,000}\times 24\times 21.295 = 259\text{ kips} \therefore \text{use } 69\text{ kips}$

Therefore in the end sections, $ØV_n$ = 0.75 (61 + 156) = 163 k

and in the center sections, $ØV_n$ = 0.75 (61 + 69) = 97 k.

At d: V_u = 92 k < ϕV_n = 163 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: V_u = 70 k < ϕV_n = 97 k, therefore the column is adequate for shear at the center section

Critical Shears in Columns at 3rd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 261 kips.

The shear capacities of the 20"x20" columns with 3 leg #4 Stirrups at 4" o.c. in the end section and 3 leg #3 Stirrups at 5" o.c. in the center section are given by:

$$ØV_n = Ø(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{261{,}000}{2{,}000\times20\times20}\right)20 \times 17.5625/1{,}000$ = 59 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times17.5625}{4\times1{,}000}$ = 158 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times17.5625}{5\times1{,}000}$ = 69 kips.

Therefore in the end sections, $ØV_n$ = 0.75 (59 + 158) = 163 k

and in the center sections, $ØV_n$ = 0.75 (59 + 69) = 96 k.

At d: V_u = 101 k < ϕV_n = 163 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: V_u = 6 k < ϕV_n = 96 k, therefore the column is adequate for shear at the center section

By inspection the remaining columns are adequate to resist the tsunami shear force.

Instead of the equivalent static load analysis performed above, it is permissible to use a non-linear analysis following the provisions of ASCE 41, or to perform a non-linear dynamic analysis of the column subjected to the debris impact strike.

B.15.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure B-16**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure B-67** and **Figure B-68** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the ends of the column, where *d* = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "d + h_c" from each end of the column, where d + h_c = 17.5 + 20 = 37.5 in.

Floor 1 – 7

End Section (A)

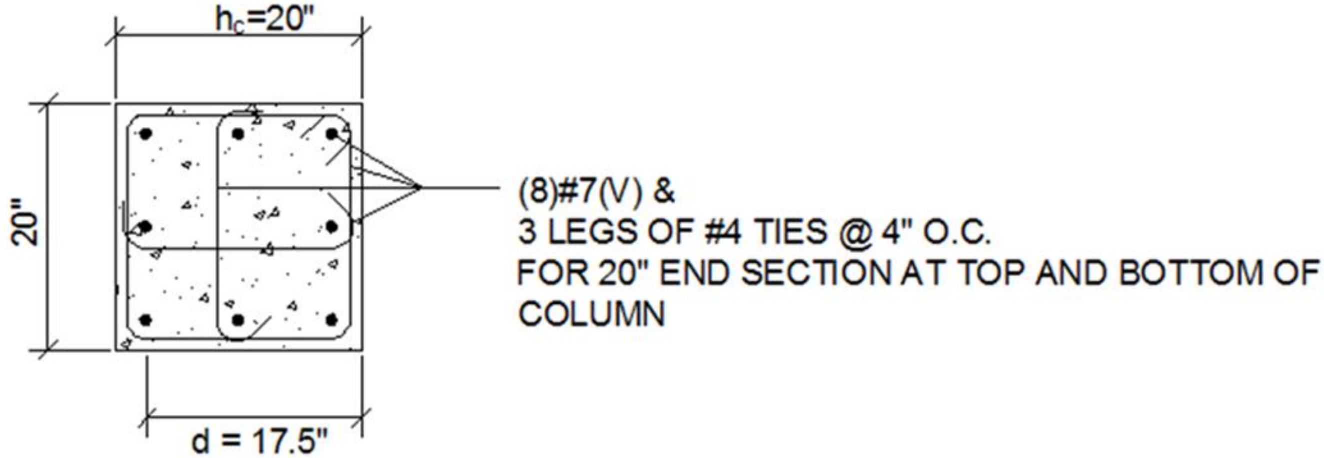

Figure B-67: Interior column, cross-section end of column at all floor levels based on SDC D design.

Center Section (B)

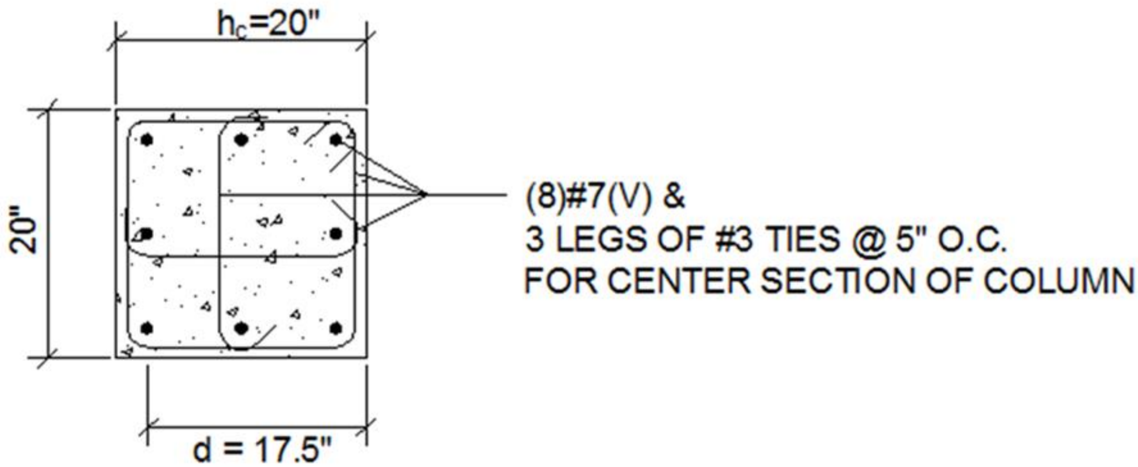

Figure B-68: Interior column, cross-section at center of column at all floor levels based on SDC D design.

B.15.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 17.83 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$$P_D = [128(14.58)(28)(6) + 123(17.83)(28) + 1.67^2(150)(66)]/1000 = 472\ k$$

Floor Live load reduction factor = $0.25+15/[4(17.83)(28)(6)]^{0.5} = 0.487$*, therefore, column base live load is:*

$P_L = 0.487[55(17.83)(28)(6)]/1000 = 80.2\ k$

Roof Live Load reduction factor = $R_1R_2 = [1.2\text{-}(0.001)(17.83)(28)](1.0) = 0.701$, *column roof live load is:*

$P_{Lr} = 0.701(20)(17.83)(28)/1000 = 6.61\ k$

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

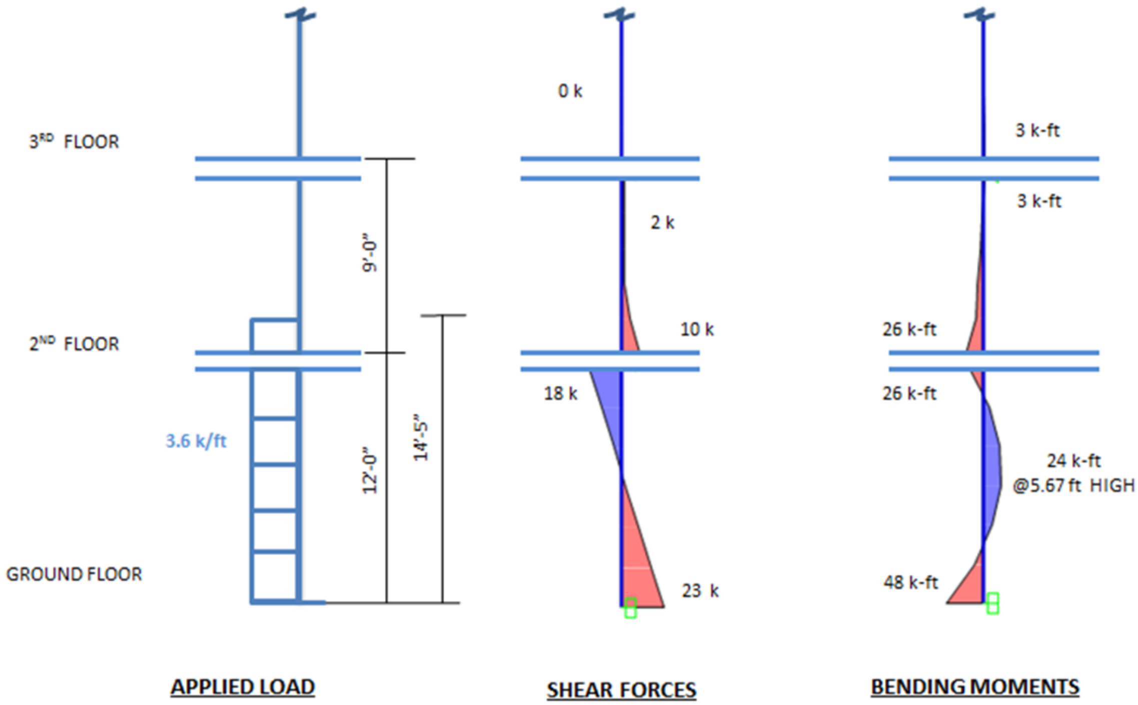

Figure B-69: Hydrodynamic loading on interior column of Monterey residential building due to Load Case 2

Table B-7 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table B-7: Results from loading conditions of Monterey residential building interior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
48	811.8	18	12	1.2D+Ftsu+0.5L (Hydro)
48	566.1	18	12	0.9D+Ftsu (Hydro)
Floor 2				
26	676.5	5	2	1.2D+Ftsu+0.5L (Hydro)
26	471.8	5	2	0.9D+Ftsu (Hydro)
Floor 3				
3	541.2	0	0	1.2D+Ftsu+0.5L (Hydro)
3	377.4	0	0	0.9D+Ftsu (Hydro)
Floor 4				
1	405.9	0	0	1.2D+Ftsu+0.5L (Hydro)
1	283.1	0	0	0.9D+Ftsu (Hydro)
Floor 5				
0	270.6	0	0	1.2D+Ftsu+0.5L (Hydro)
0	188.7	0	0	0.9D+Ftsu (Hydro)
Floor 6				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.4	0	0	0.9D+Ftsu (Hydro)
Floor 7				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.4	0	0	0.9D+Ftsu (Hydro)

B.15.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure B-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure B-70 shows the interaction diagram for the typical exterior column including the tsunami load combinations.

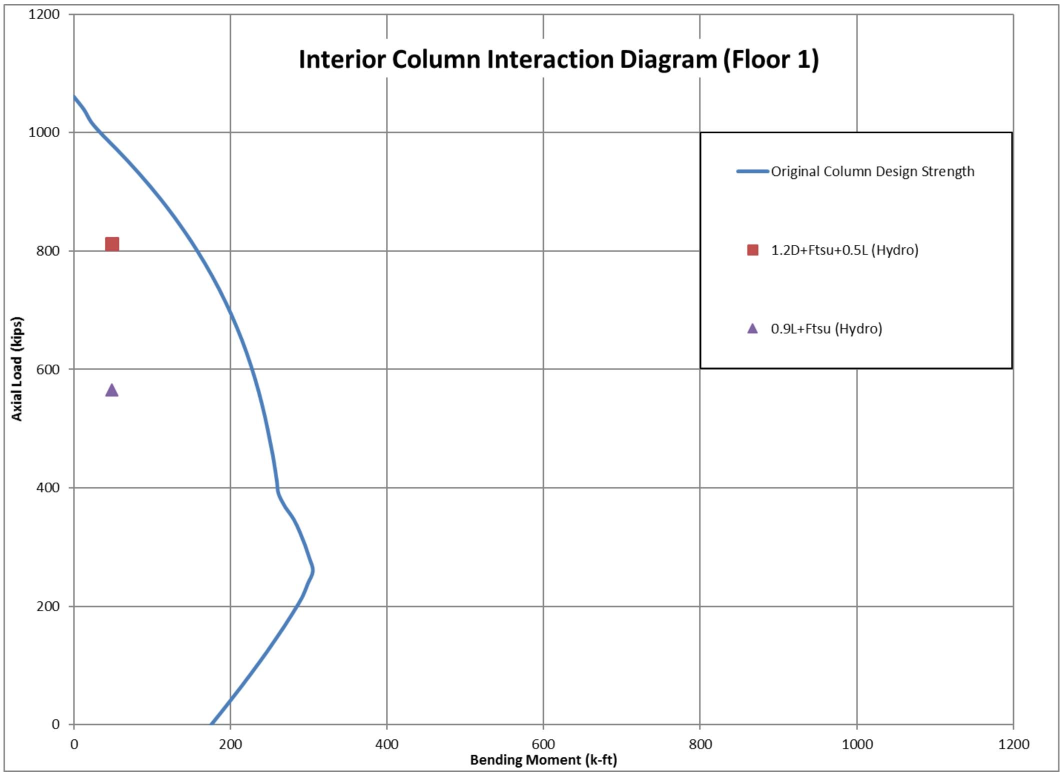

Figure B-70: Interaction diagram for typical ground floor residential interior column showing tsunami load combinations

B.15.2.3 Interior Column Shear Design

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of ($1.2D + F_{TSU} + 0.5L$) per **Eqn. 6.8.3.3.-1a,** P_u = 811.8 kips.

The shear capacities of the existing 20"x20" column with 3 leg #4 Stirrups @ 4" o.c. in the end sections and 3 leg #3 Stirrups @ 5" o.c. in the center section are given by:

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{811{,}800}{2{,}000\times 20\times 20}\right)20\times 17.5625/1{,}000$ = 90 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.2)\times 60{,}000\times 17.5625}{4\times 1{,}000}$ = 158 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.11)\times 60{,}000\times 17.5625}{5\times 1{,}000}$ = 70 kips

Therefore in the end sections, $\text{Ø}V_n = 0.75\ (90 + 158) = 186$ k

and in the center sections, $\varnothing V_n$ = 0.75 (90 + 70) = 120 k

At d: V_u = 37 k < ϕV_n = 186 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 24 k < ϕV_n = 120 k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear force.

B.15.3 Typical Exterior Wall Design

A section of exterior wall along Grid Line D from Figure A-16 adjacent to the mechanical room was analyzed. The wall is part of the lateral resisting system for seismic loads, acting as a shear wall for longitudinal forces and boundary element for transverse forces. Seismic Design Category D design and detailing of the 10" thick wall resulted in the reinforcement layout shown in **Figure B-71** to **Figure B-73**. The wall will now be checked for tsunami loads.

For comparative purposes with the debris impact loads, the ultimate shear forces and bending moments are provided for an effective width of wall equal to 5.67 ft. The critical shear force occurs at a distance "*d*" from the base of the wall, where *d* = 10-0.75-1"/2 = 8.75 in.

Floor 1

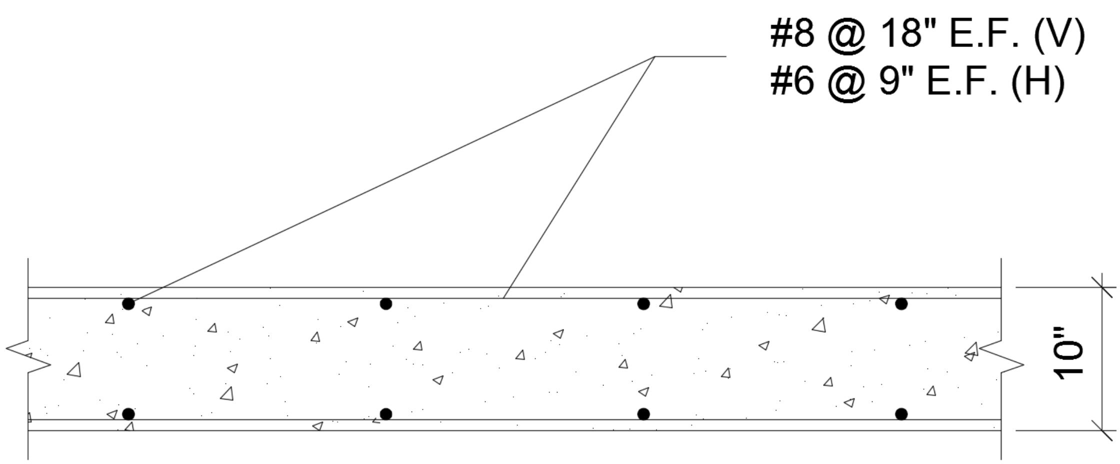

Figure B-71: Segment of exterior wall cross-section at the 1st floor level based on SDC D design.

Floor 2

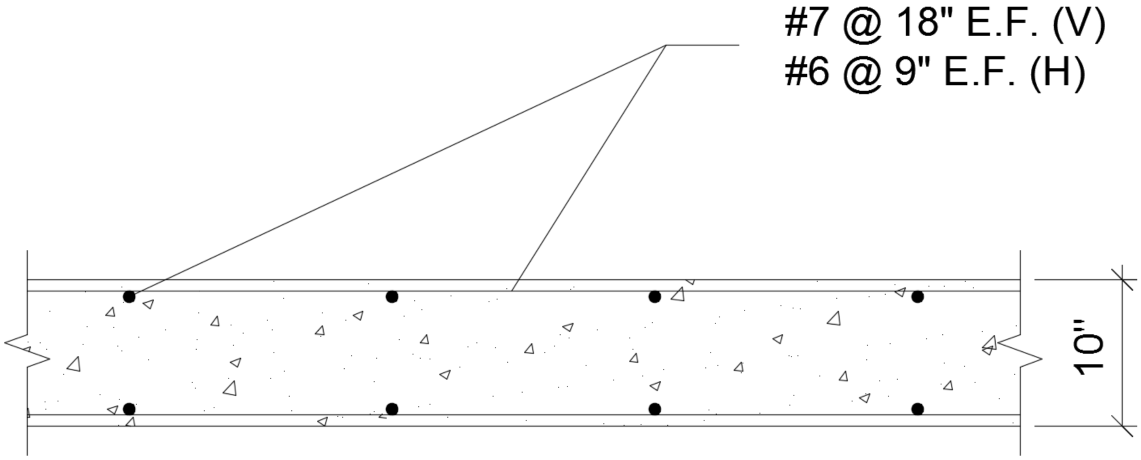

Figure B-72: Segment of exterior wall cross-section at the 2nd floor level based on SDC D design.

Floor 3 – 7

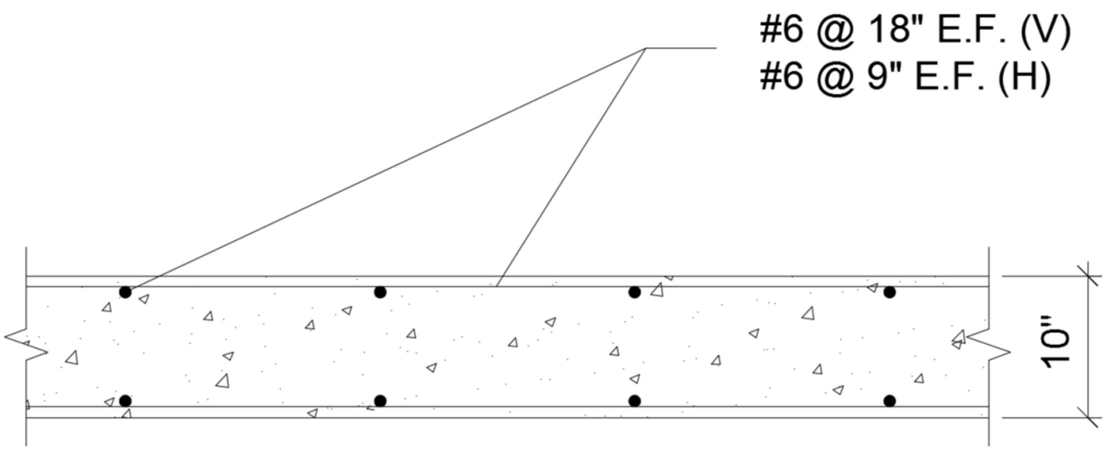

Figure B-73: Segment of exterior wall cross-section at the 3rd – 7th floor level based on SDC D design.

B.15.3.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 5.5 ft. Gravity loads will be computed per foot width of wall.

The Dead Load at the base of the wall is:

$$P_D = [128(5.5)(1)(6)+123(5.5)(1)+0.83(1)(150)(66)]/1000 = 13.1 \text{ k/ft}$$

Floor Live load reduction: Reduction Factor = $0.25+15/[1(5.5)(28)(6)]^{0.5} = 0.743$

$$P_L = 0.743[55(5.5)(1)(6)]/1000 = 1.35 \text{ k/ft}$$

Roof Live Load reduction: Reduction Factor = $R_1R_2= [1.2-(0.001)(5.5)(28)](1.0) = 1.05$*, Use 1.0*

$P_{Lr} = 20(5.5)(1)/1000 = 0.110\ k/ft$

The wall is assumed to span vertically between floors because of the 28' width compared with the 12' height of the second floor. The wall must resist both of the hydrodynamic pressures computed in Section B.13.2.

For Load Case 2, Eqn. 6.10-5a applied with a resulting hydrodynamic pressure of 2183 psf acting over the bottom 14.4 ft of the wall. For a 5.67 foot width of wall this results in a uniformly distributed load of 5.67 x 2183/1000 = 12.4 k/ft.

For the bore matching 1/3 of the wall width, Eqn. 6.10-5b resulted in a hydrodynamic pressure of 2660 psf acting over the bottom 9.33 ft of the wall. For a 5.67 foot width of wall this results in a uniformly distributed load of 5.67 x 2660/1000 = 15.1 k/ft.

Analysis of a 5.67 foot width of wall with the applied tsunami hydrodynamic loads from Load Case 2 results in the following bending moment and shear force diagrams:

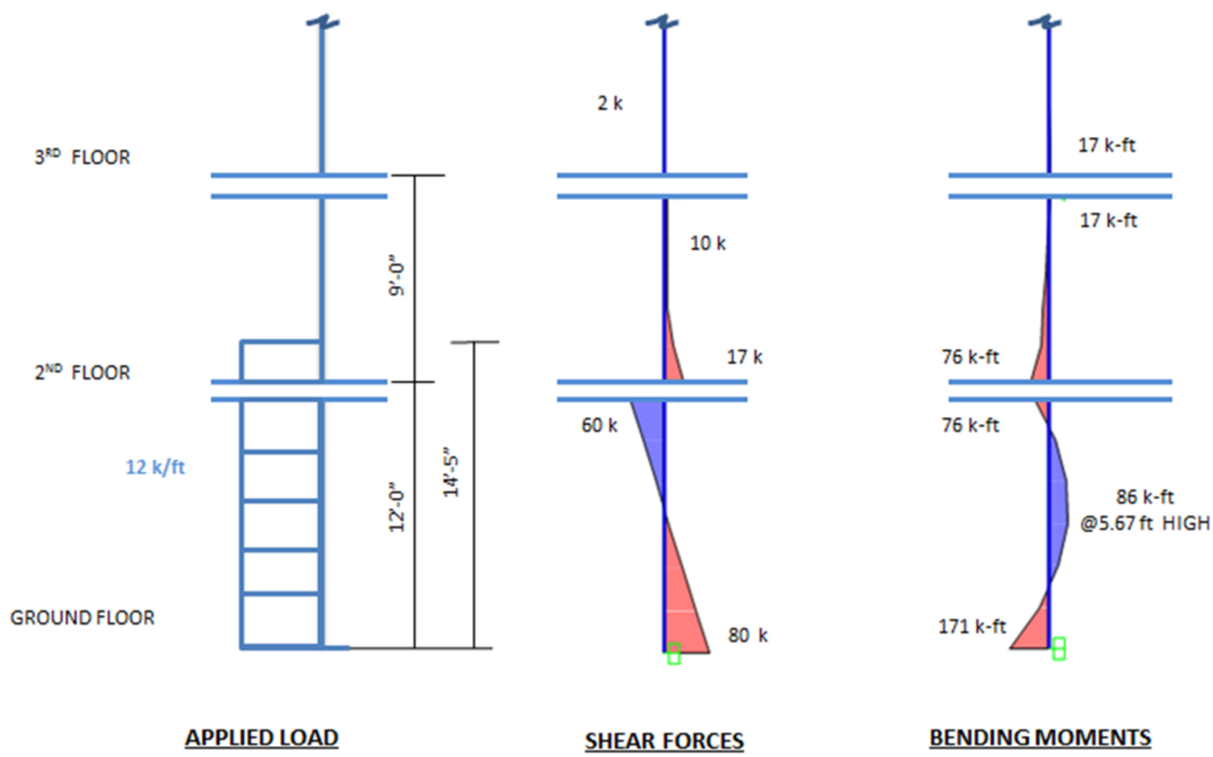

Figure B-74: Hydrodynamic loading on exterior wall of Monterey residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike, acts over an effective width of 5.67 ft, at a point just below the slab at each inundated floor for maximum shear and at the mid-height of the clear column height for maximum bending moments. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure B-75** to **Figure B-77**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown in **Figure B-78** to **Figure B-80**.

Impact load at d:

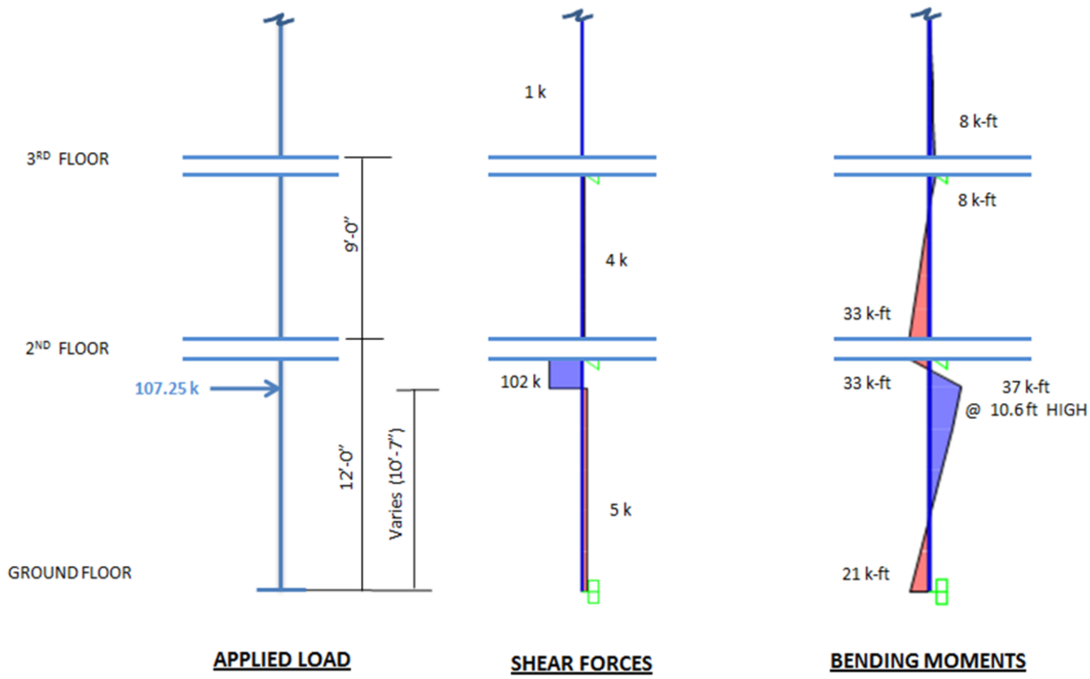

Figure B-75: Impact load applied at d away from the end of column on the ground floor

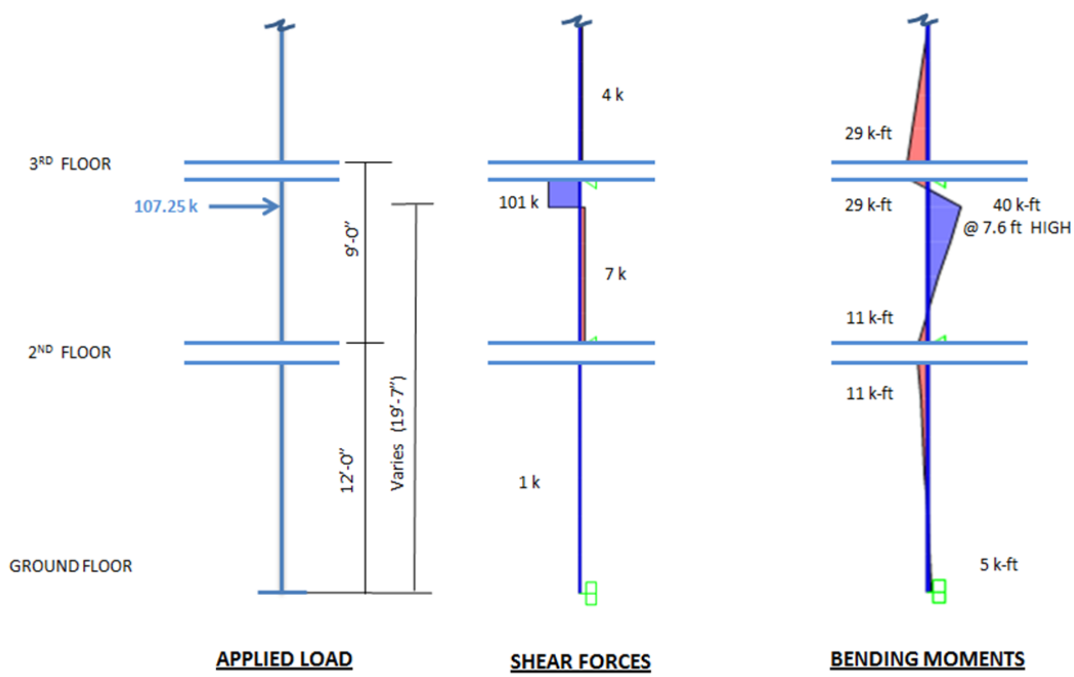

Figure B-76: Impact load applied at d away from the end of column on the 2nd floor

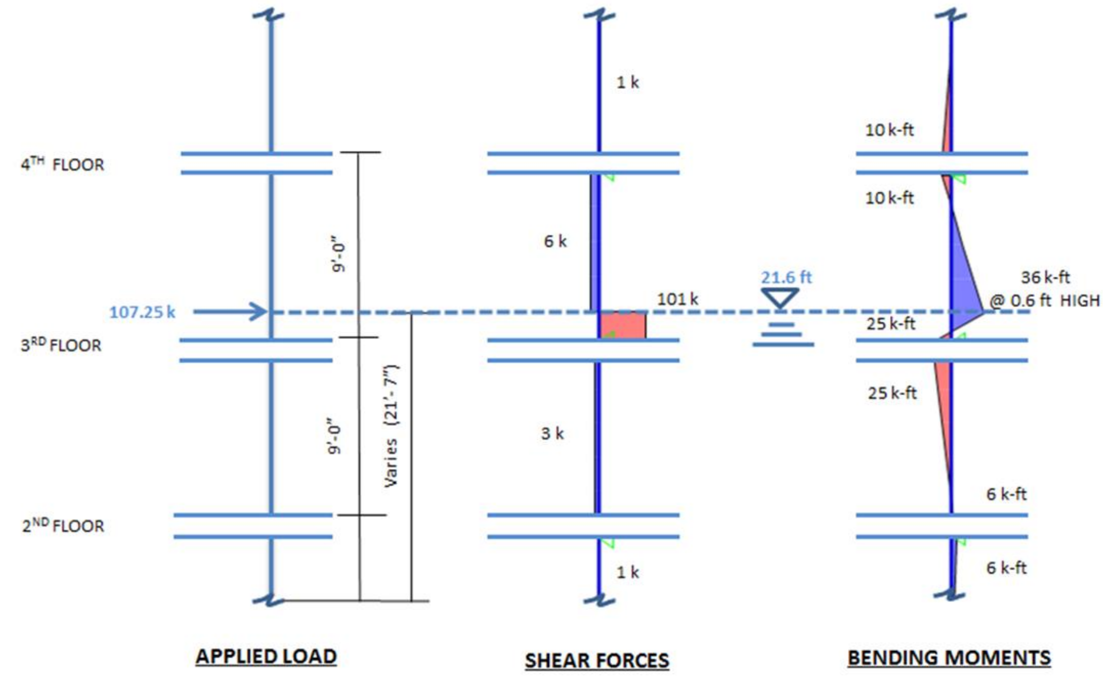

Figure B-77: Impact load applied at 0.6 ft away instead of "d" as water level is lower than "d" away from the end of column on the 3rd floor

Impact load at mid-heights:

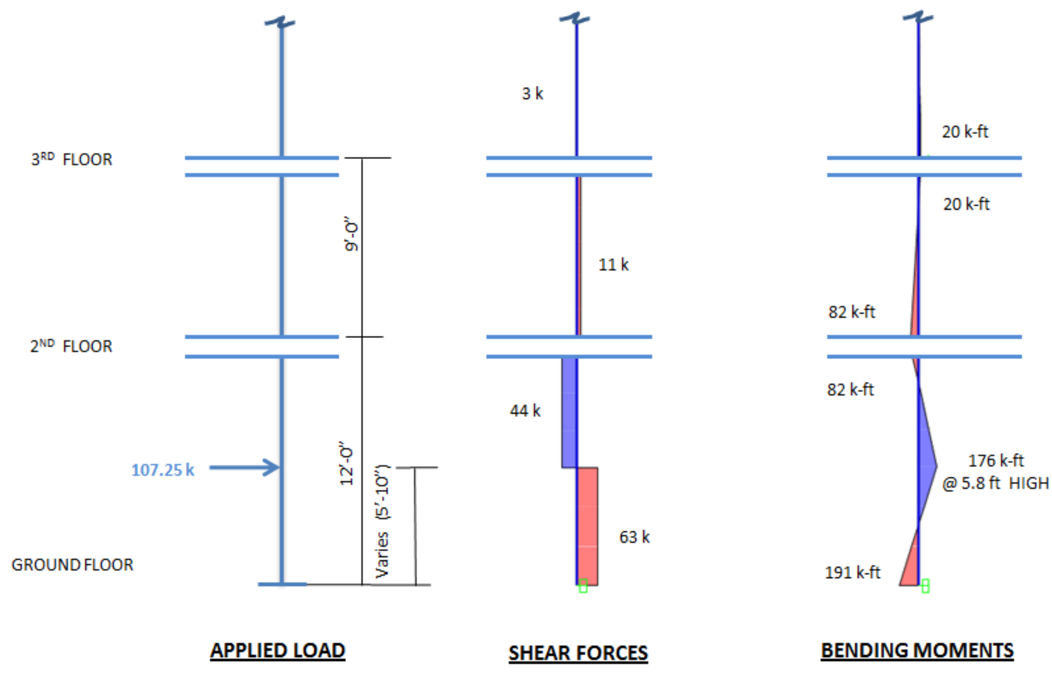

Figure B-78: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

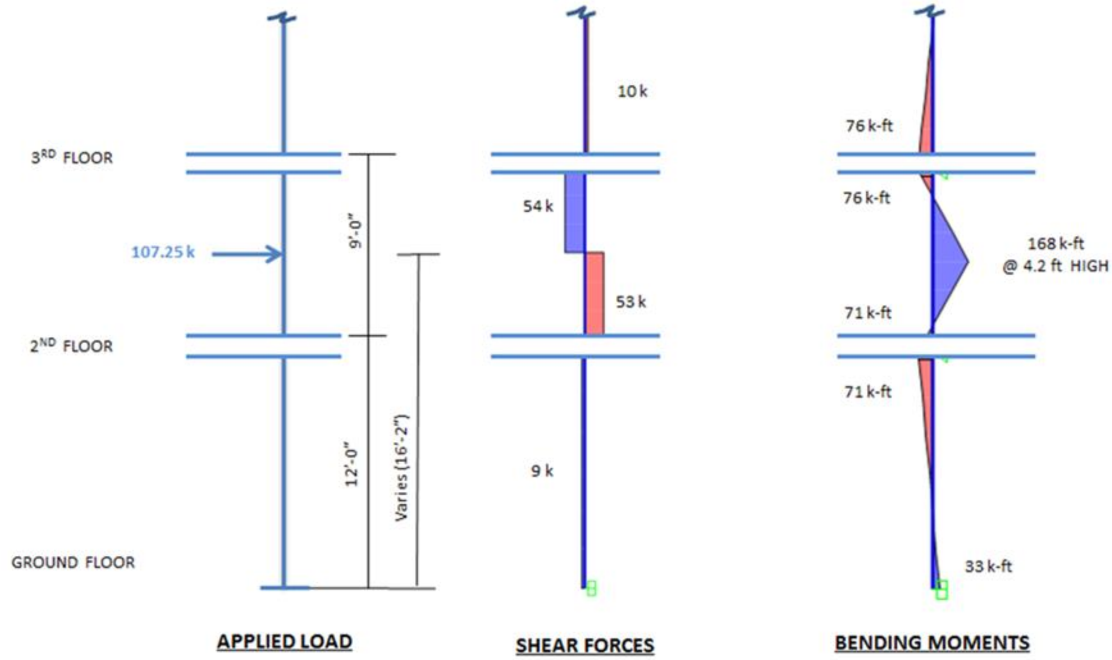

Figure B-79: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

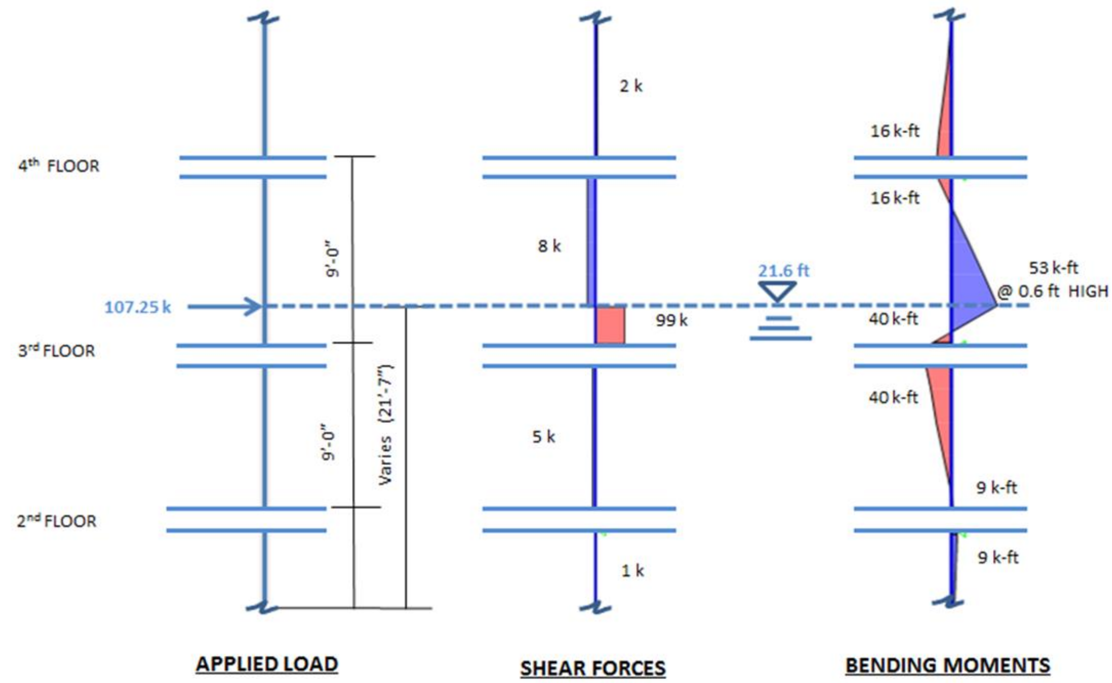

Figure B-80: Impact load applied at 0.6 ft away instead of mid-height as water level is lower than mid-height away from the end of column on the 3rd floor

Table B-8: Results from loading conditions of Monterey residential building exterior shear wall

Moment K-ft	Axial Load Kips	Shear @ d Kips	Load Combination
Floor 1			
171	92.91	71	1.2D+Ftsu+0.5L (Hydro)
171	6.89	71	0.9D+Ftsu (Hydro)
176	92.91	102	1.2D+Ftsu+0.5L (Impact)
176	6.89	102	0.9D+Ftsu (Impact)
Floor 2			
76	79.63	14	1.2D+Ftsu+0.5L (Hydro)
76	5.90	14	0.9D+Ftsu (Hydro)
168	79.63	101	1.2D+Ftsu+0.5L (Impact)
168	5.90	101	0.9D+Ftsu (Impact)
Floor 3			
17	66.36	2	1.2D+Ftsu+0.5L (Hydro)
17	4.92	2	0.9D+Ftsu (Hydro)
53	66.36	101	1.2D+Ftsu+0.5L (Impact)
53	4.92	101	0.9D+Ftsu (Impact)
Floor 4			
4	53.09	1	1.2D+Ftsu+0.5L (Hydro)
4	3.93	1	0.9D+Ftsu (Hydro)
16	53.09	1	1.2D+Ftsu+0.5L (Impact)
16	3.93	1	0.9D+Ftsu (Impact)
Floor 5			
1	39.82	0	1.2D+Ftsu+0.5L (Hydro)
1	2.95	0	0.9D+Ftsu (Hydro)
4	39.82	0	1.2D+Ftsu+0.5L (Impact)
4	2.95	0	0.9D+Ftsu (Impact)
Floor 6			
0	26.54	0	1.2D+Ftsu+0.5L (Hydro)
0	1.97	0	0.9D+Ftsu (Hydro)
1	26.54	0	1.2D+Ftsu+0.5L (Impact)
1	1.97	0	0.9D+Ftsu (Impact)
Floor 7			
0	13.27	0	1.2D+Ftsu+0.5L (Hydro)
0	0.98	0	0.9D+Ftsu (Hydro)
0	13.27	0	1.2D+Ftsu+0.5L (Impact)
0	0.98	0	0.9D+Ftsu (Impact)

B.15.3.2 Existing Exterior Shear Wall Design for Combined Gravity and Tsunami Loads

The shear wall chosen at Grid Line D from **Figure B-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**.

Figure B-81 to **Figure B-83** shows the interaction diagrams for the typical exterior shear wall including the tsunami load combinations.

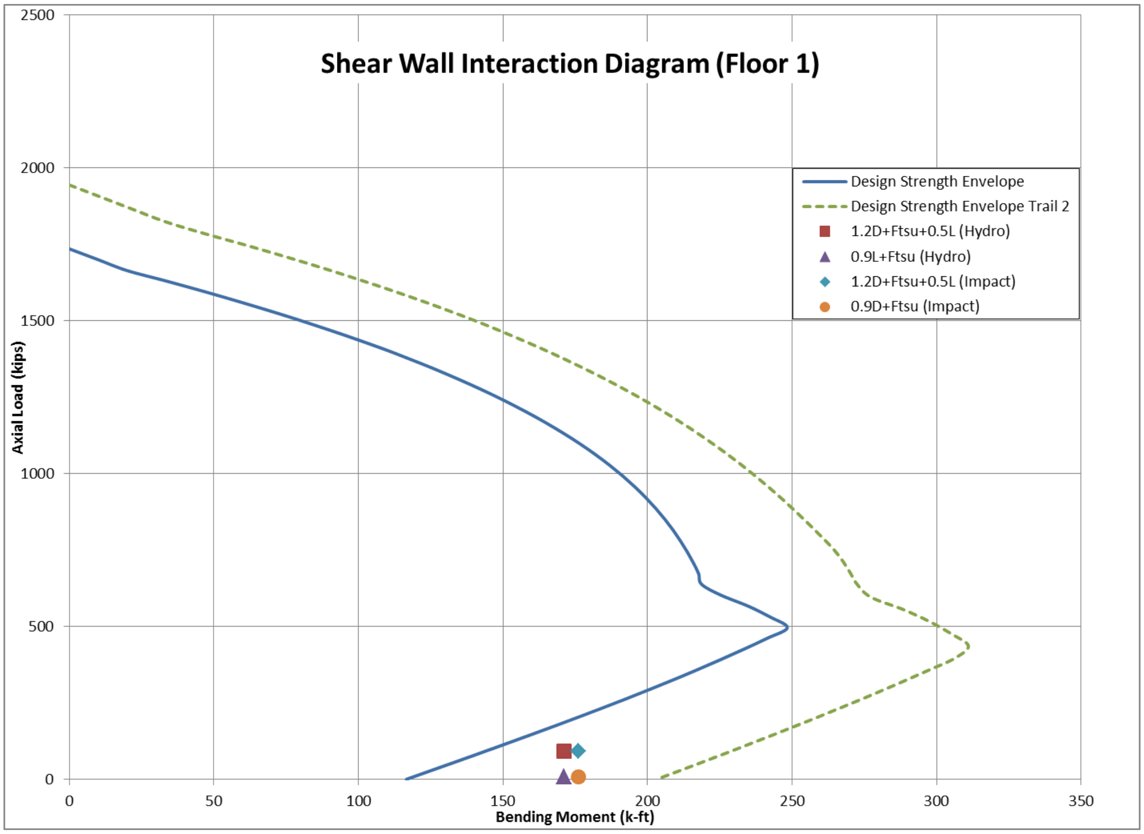

Figure B-81: Interaction diagram for typical ground floor exterior wall segment showing tsunami load combinations

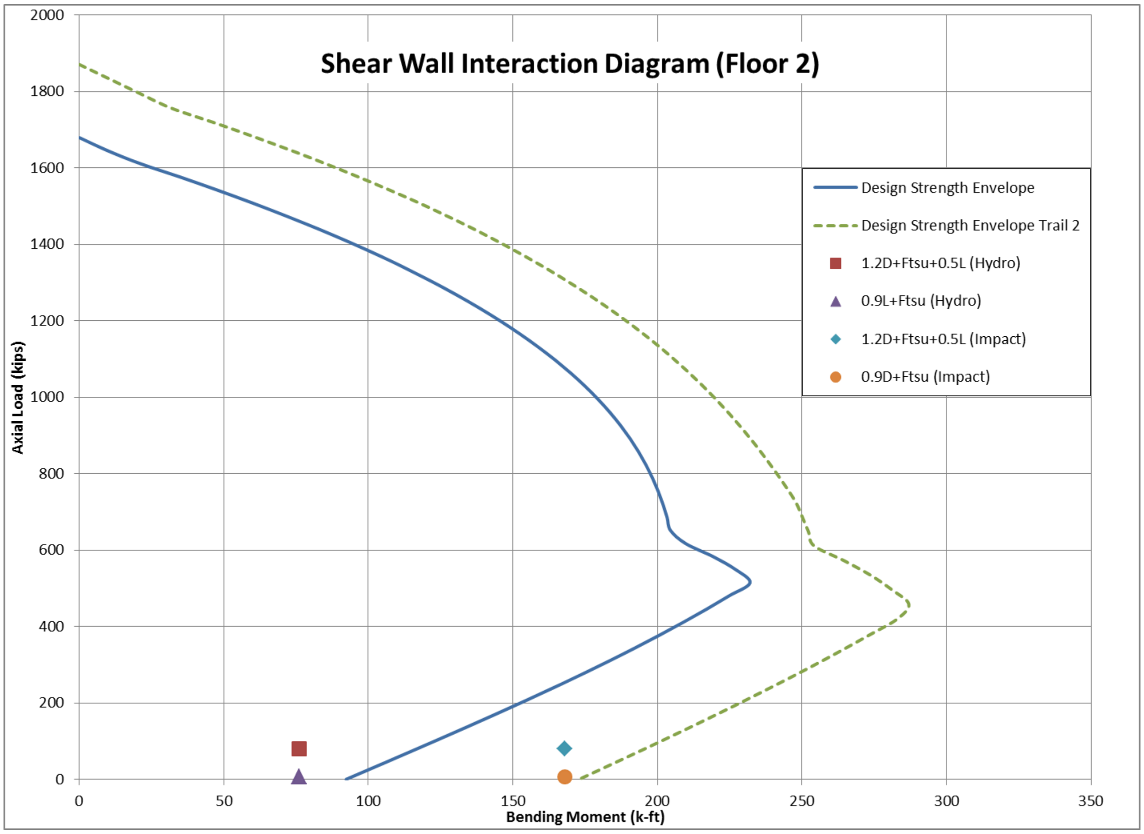

Figure B-82: Interaction diagram for typical 2^{md} floor exterior wall segment showing tsunami load combinations

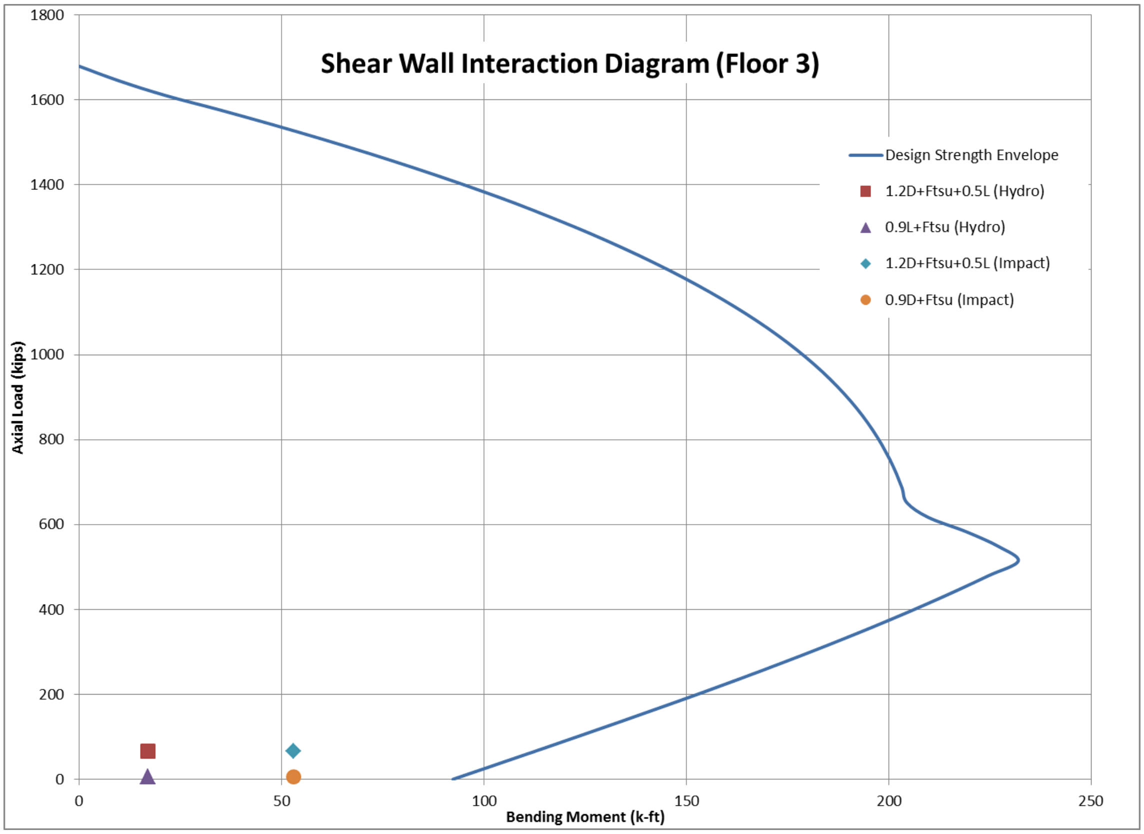

Figure B-83: Interaction diagram for typical 3rd floor exterior wall segment showing tsunami load combinations

By inspection the remaining shear walls are adequate to resist the tsunami bending moments.

B.15.3.3 New Typical Shear Wall Design

The interaction diagrams show that the walls on floors 1 to 4 are inadequate for the bending moments due to hydrodynamic load, while those at levels 3 to 6 are inadequate for bending moments resulting from derbies impact. **Figure B-84** to **Figure B-86** show the revised wall designs required to resist the tsunami loads. **Figure B-87** shows the side view of the wall with shear stud rails included.

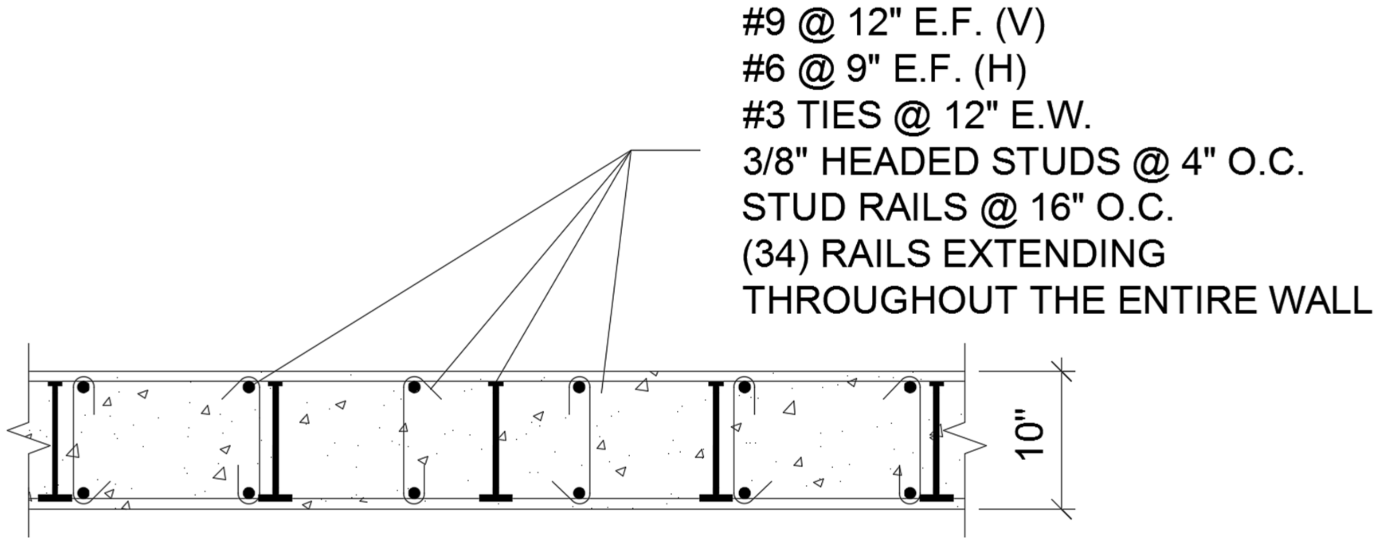

Figure B-84: New exterior wall, cross-section at the ground floor level based on tsunami design requirements.

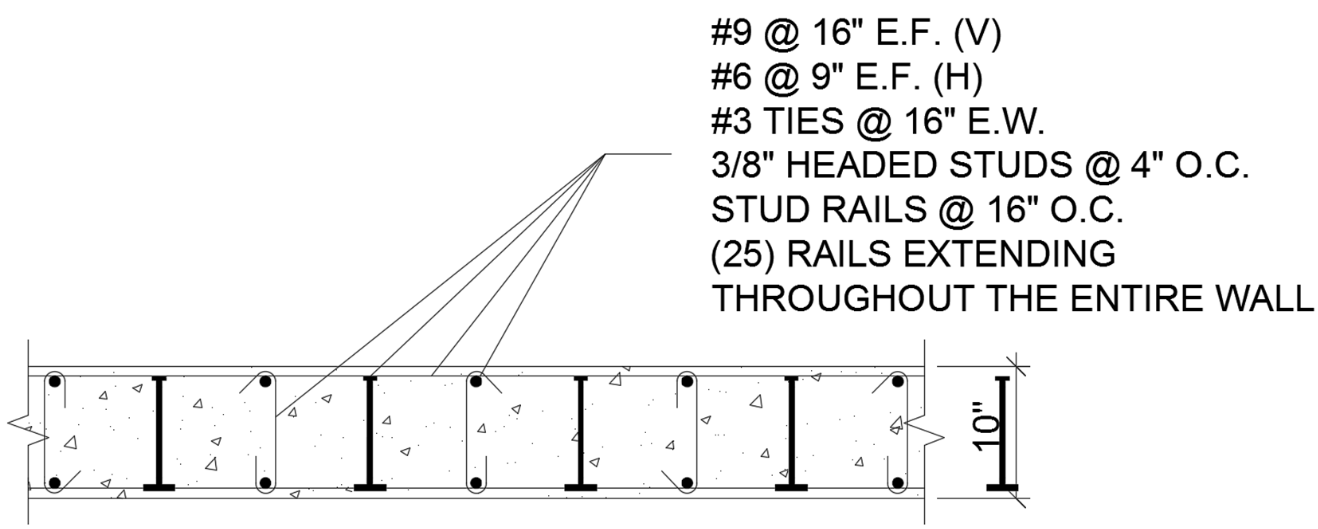

Figure B-85: New exterior wall, cross-section at the 2nd floor level based on tsunami design requirements.

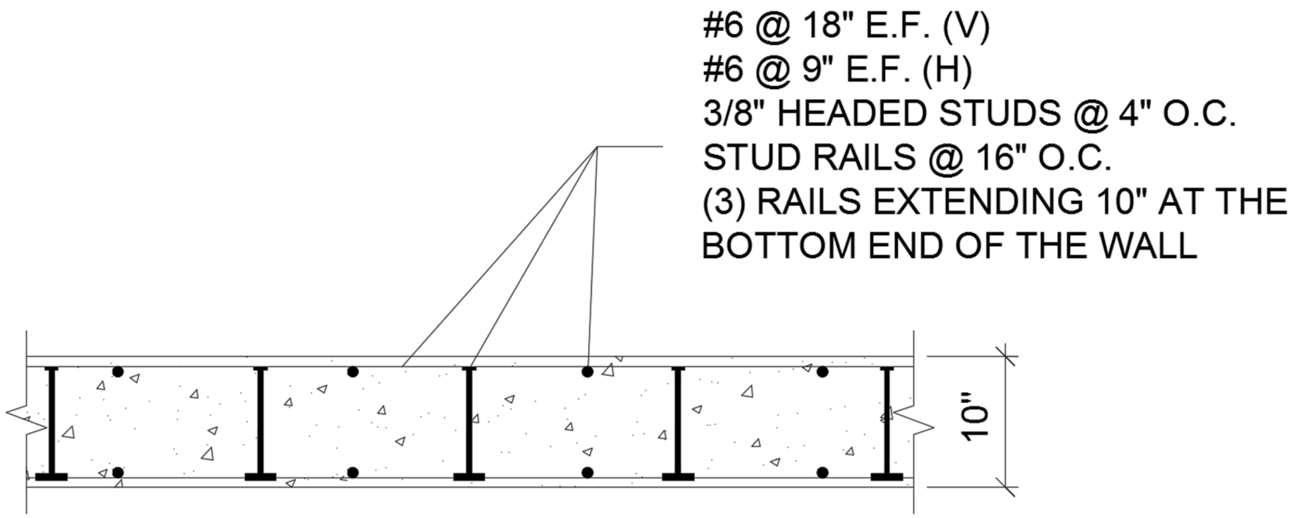

Figure B-86: New exterior wall, cross-section at the 3rd floor level based on tsunami design requirements.

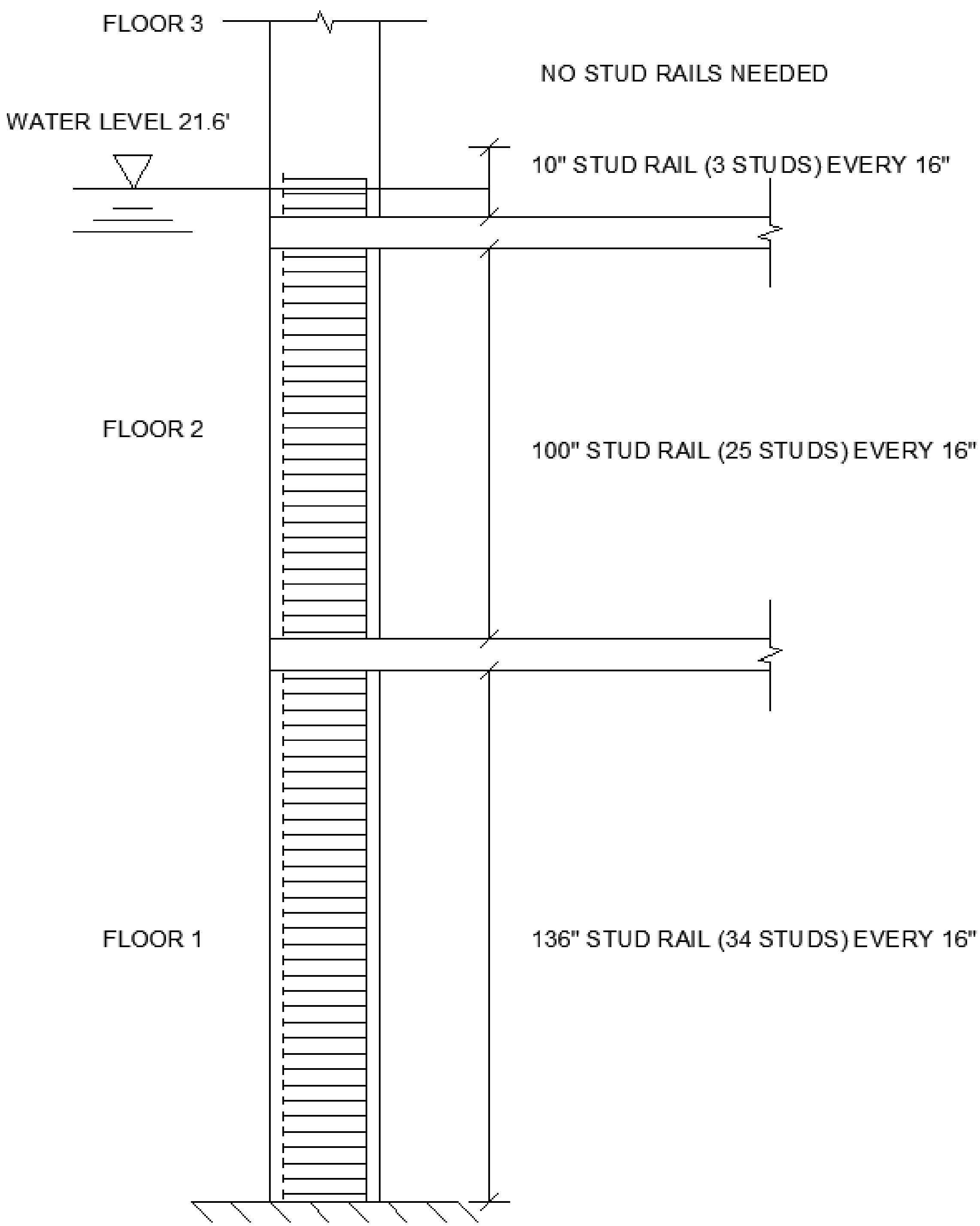

Figure B-87: Stud Rail Diagram for the Floor 1 – 3

B.15.3.4 Exterior Shear Wall Shear Design

Critical Shears:

1st Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$), P_u = 6.89 k:

Shear Capacity of existing shear wall (10” thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{836}{2{,}000 \times 68 \times 10}\right)\,1\sqrt{4000}\ \times 68 \times 8.75/1{,}000$ = 75 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{6.89 \times 1{,}000}{580}\right) \times 68 = 836\ lb$

I_w = 28’+11’+11’- 2 x 10” (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (75+0) = 56 kips

V_{tsu} = 102 kips > $ØV_n$ = 56 kips ∴ 61 kips needed

Shear Capacity for New Shear Wall (10” thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{836}{2{,}000 \times 68 \times 10}\right)\,1\sqrt{4000}\ \times 68 \times 8.75/1{,}000$ = 75 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{6.89 \times 1{,}000}{580}\right) \times 68 = 836\ lb$

I_w = 28’+11’+11’- 2 x 10” (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (75+0) = 56 kips

V_{tsu} = 102 kips > $ØV_n$ = 56 kips ∴ 61 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.75}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"} = 4.25$

A_v = 3/8” stud = 0.11 in^2

s_{max} = d/2 = 8.75/2 = 4.375 in

$s_{needed} = \dfrac{n \times A_v \times f_y \times d}{V_{S\,(needed)}} = \dfrac{4.25 \times 0.11 \times 60 \times 8.75}{61} = 4.1$ in

∴ $s_{used} = 4$ in

$ØV_n = Ø(V_C + V_S) = 0.75\ (75 + 62) = 103$ Kips

$ØV_n = 103$ Kips > $V_{Tsu} = 102$ Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ mid\text{-}height} = 63$ Kips > $ØV_c = 56$ Therefore the rails go up the entire wall of the Shear Wall

2nd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$), $P_u = 5.9$ k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{717}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.8125/1{,}000 = 76$ kips

$N_u = \left(\frac{P_u \times 1{,}000}{I_w}\right) \times b = \left(\frac{5.9 \times 1{,}000}{580}\right) \times 68 = 717\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76+0) = 57$ kips

$V_{tsu} = 101$ kips > $ØV_n = 57$ kips ∴ 59 kips needed

Shear Capacity for New Shear Wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{717}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.8125/1{,}000 = 76$ kips

$N_u = \left(\frac{P_u \times 1{,}000}{I_w}\right) \times b = \left(\frac{5.9 \times 1{,}000}{580}\right) \times 68 = 717\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76+0) = 57$ kips

$V_{tsu} = 101$ kips > $ØV_n = 57$ kips ∴ 59 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.8125}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"}$ = 4.25

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 = 8.8125/2 = 4.41 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{4.25 \times 0.11 \times 60 \times 8.8125}{59}$ = 4.2 in

∴ s_{used} = 4 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76 + 62) = 103 Kips

$ØV_n$ = 103 Kips > V_{Tsu} = 101 Kips Therefore the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ mid\text{-}height}$ = 54 Kips < $ØV_c$ = 57 Therefore the rails go up the entire wall of the Shear Wall

3rd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 4.92 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{577}{2,000\times68\times10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1,000$ = 76 kips

$N_u = (\frac{P_u\times1,000}{I_w}) \times b = \left(\frac{4.92\times1,000}{580}\right) \times 68 = 577\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_c + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 101 kips > $ØV_n$ = 57 kips ∴ 58 kips needed

Shear Capacity for New Shear Wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{577}{2,000\times68\times10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1,000$ = 76 kips

$N_u = (\frac{P_u\times1,000}{I_w}) \times b = \left(\frac{4.92\times1,000}{560}\right) \times 68 = 577\ lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76 + 0) = 57 kips

V_{tsu} = 101 kips > $ØV_n$ = 57 kips ∴ 58 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.875}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"} = 4.25$

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =8.875/2 = 4.44 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{4.25 \times 0.11 \times 60 \times 12.875}{58}$ = 4.3 in

∴ s_{used} = 4 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (78 + 62) = 105 Kips

$ØV_n$ = 104 Kips > V_{Tsu} = 101 Kips Therefore the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 10"}$ = 8 Kips < $ØV_c$ = 53 Therefore rails go up 10" (3 Studs) at the bottom end of the Shear Wall

4th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 3.93 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{461}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u \times 1{,}000}{l_w}) \times b = \left(\frac{3.93 \times 1{,}000}{580}\right) \times 68 = 461\ lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 1 kips < $ØV_n$ = 57 kips ∴ no shear studs ae needed

By inspection the remaining shear walls are adequate to resist the tsunami shear force.

C. Hilo Design Example

C.1 Project Site

The Hilo design example considers a multi-story reinforced concrete building in Hilo, Hawaii, at the location shown in **Figure C-1**. The center of the building footprint is located at 19.720867 N; 155.083286 W, which is 1101 feet from the shoreline. **Figure C-1** also shows the three topographic transects along which the Energy Grade Line Analysis needs to be applied. The center transect, C, is drawn perpendicular to the shoreline, represented by the average coastline for 500 feet either side of the center transect. The clockwise, CW, and counterclockwise, CCW, transects are generated by rotating the center transect through 22.5 degrees in each direction, about the geometric center of the building plan at the grade plane (ASCE 7 Section 6.8.6.1). Each transect is then extended till it reaches the runup points on the ASCE 7 Tsunami Design Zone map. If the end of a transect falls between two of the runup points, than the runup elevations can be interpolated. The resulting runup elevations for each transect are given in **Table C-1** along with the approximate inundation limit distances obtained using Google Earth. These inundation limit distances will be revised once the runup elevations are plotted on the respective topographic profiles.

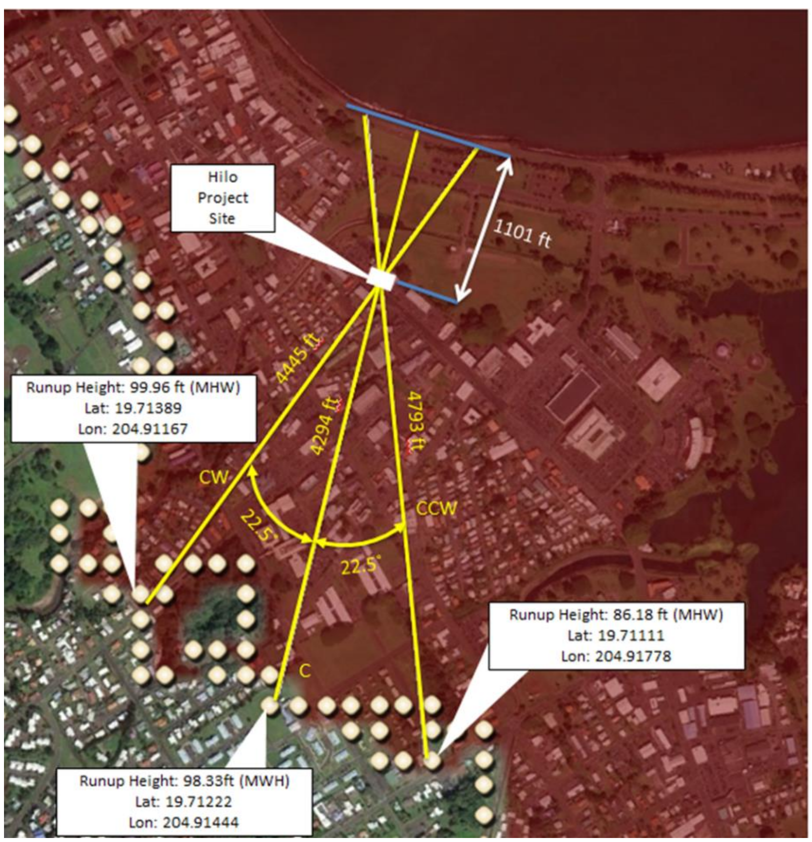

Figure C-1: Location of project site in Hilo, Hawaii, relative to inundation line defined by ASCE7-16 Tsunami Design Zone Map. The 22.5° variation in principal flow direction required by Section 6.8.6.1 results in Clockwise (CW) and Counterclockwise (CCW) transects on either side of the Center (C) transect.

Table C-1: Runup elevation and inundation limits for three transects through the Hilo Project site.

Transect	Runup Elevation (ft)		Inundation Limit (ft)	
	MHW Reference		From Google Earth	From WGS 84 Transect
	From TDZ	Incl. Sea Level Rise		
Center	**98.33**	**98.86**	**4278**	**4294**
Counterclockwise	**86.18**	**86.71**	**4761**	**4793**
Clockwise	**99.96**	**100.49**	**4315**	**4445**

C.2 Sea Level Change – Section 6.5.3

ASCE 7 Section 6.5.3 requires that any anticipated sea level rise be included in the runup elevation used in the tsunami design. For this example, we will assume sea level change based on a 50 year project life cycle. ASCE 7 Commentary Section C6.5.3 provides a link to http://tidesandcurrents.noaa.gov/sltrends for historical sea level trends relative to mean sea level (MSL).

From the referenced website the following information is obtained:

"Hilo, Hawaii; 1617760

The mean sea level trend is 2.95 mm/year with a 95% confidence interval of +/- 0.31 mm/year based on monthly mean sea level data from 1927 to 2015 which is equivalent to a change of 0.97 feet in 100 years."

The tsunami design should therefore consider the extrapolated prediction of 3.26 mm/year over the 50 year project life cycle. This results in a sea level rise of 163 mm or 6.42" (0.534 ft). This must be added to the runup elevation for use in the Energy Grade Line Analysis, as shown in **Table C-1**.

C.3 Topographic Profiles

The topographic profiles along each of these transects was obtained from a Digital Elevation Model, DEM, with the following datums and resolution:

Horizontal Datum: WGS 84

Vertical Datum: MHW

Resolution: 1/3 sec (approximately 10)

The topographic profiles are shown for the Center, Counterclockwise and Clockwise transects in **Figure C-2**, **Figure C-3**, and **Figure C-4** respectively. A horizontal line is plotted on each profile representing the runup elevation (including sea level rise) for each of these transects relative to the MHW datum from **Table C-1**. The point where this line intersects the profile represents the inundation limit and the starting point for the Energy Grade Line Analysis. The resulting inundation limit should be cross-checked with the Tsunami Design Zone map inundation line to ensure that they are similar distances from the shoreline (See **Table C-1**). If the TDZ inundation is significantly greater than the first intersection of the runup elevation line with the topographic profile, it may indicate that a region of high ground is present in the inundation zone. The runup elevation must then be modified to match this high ground elevation and the corresponding inundation limit determined where the modified runup elevation next intersects the topographic profile. The resulting values for inundation limit are shown in **Table C-1** and are used in the EGLA along each transect.

The project site location is also indicated on each plot. For the center transect, the site is located 1,101 feet from the shoreline (**Figure C-2**). For the other transects the distance from the shoreline is slightly longer. The elevations at the project site vary slightly for the three transects, which can be attributed to slight differences in the elevation data points used to generate each transect profile.

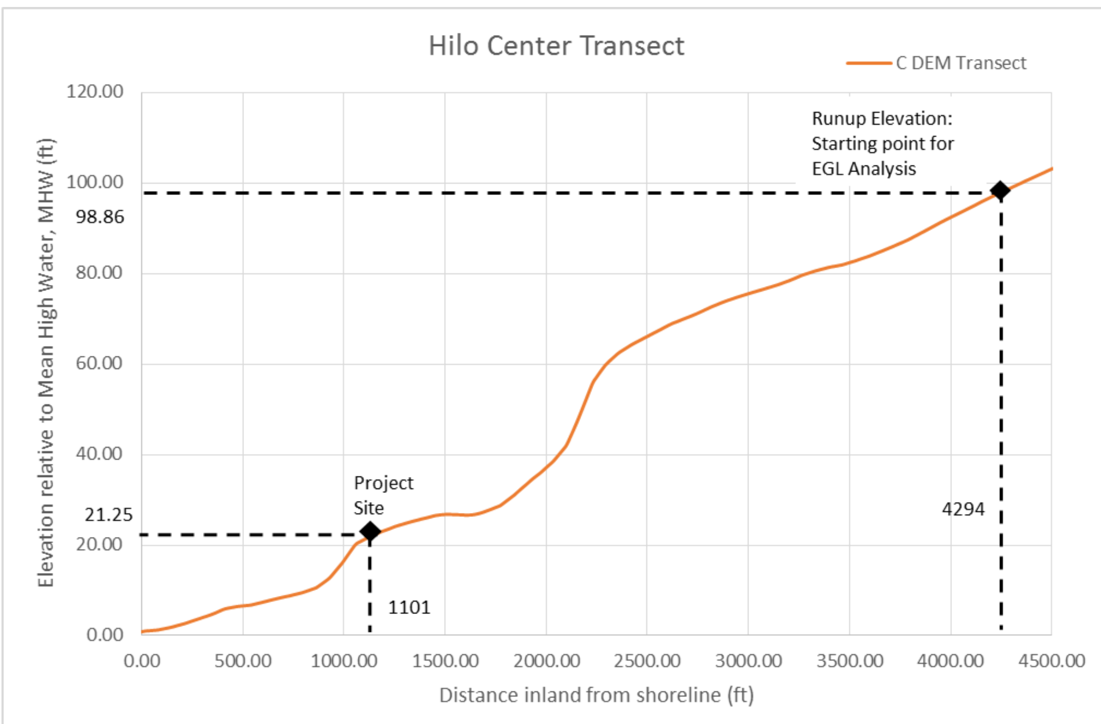

Figure C-2: Topographic profile for Center transect

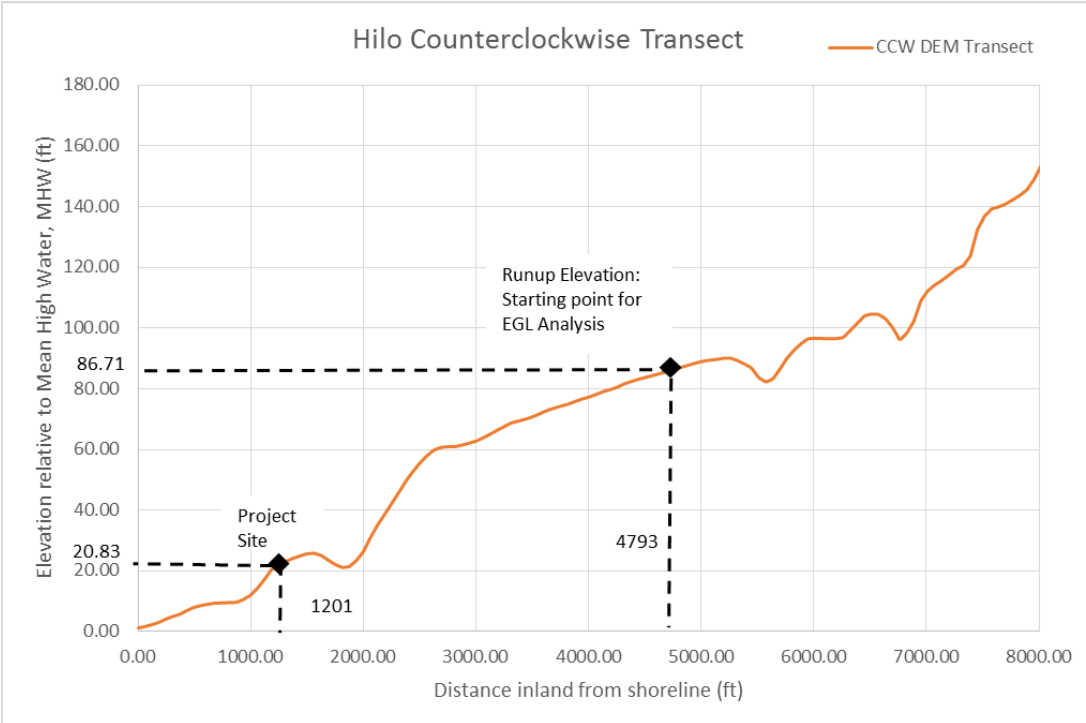

Figure C-3: Topographic profile for Counterclockwise transect

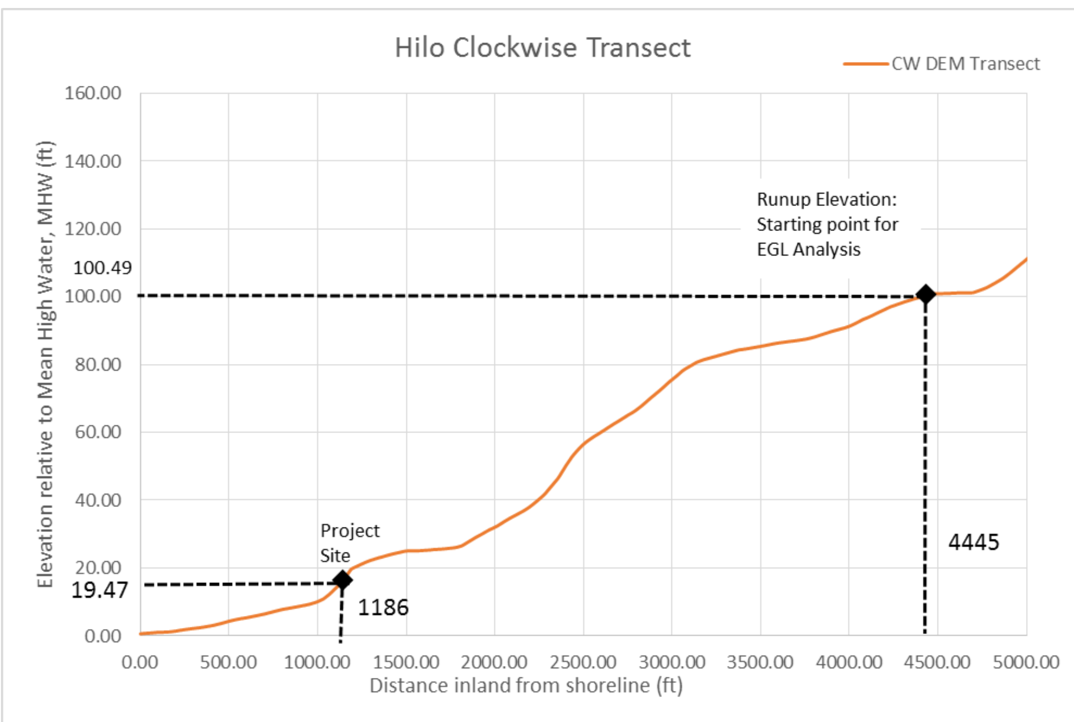

Figure C-4: Topographic profile for Clockwise transect

C.4 Tsunami Bore Determination

In order to determine whether or not a tsunami bore must be considered at the project site, the conditions in ASCE 7 Section 6.6.4 are evaluated for each transect. Tsunami bores shall be considered where any of the following conditions exist:

8. Prevailing nearshore bathymetric slope is 1/100 or milder – NO (See **Figure C-5** and associated discussion).
9. Shallow fringing reefs or other similar step discontinuities – YES
10. Where historically documented – YES.
11. As described in the Recognized Literature – Does not apply.
12. As determined by a site-specific inundation analysis – not required for TRC II buildings.

Therefore bore loading must be considered in this design.

Figure C-5 shows the approach to determining the average nearshore bathymetric slope so as to determine whether or not tsunami bores need to be considered per **ASCE 7 Section 6.6.4**. A central line is drawn perpendicular to the shoreline. This line is an extension of the center transect running through the project site. The distance from the shoreline to the 100 meter bathymetric line, indicated by the offshore data points in the ASCE offshore wave maps, is then used to determine the average nearshore bathymetric slope. If any of the transect lines does not intersect the 100 meter bathymetric line, this transect can be ignored for the purpose of determining whether or not there is a bore.

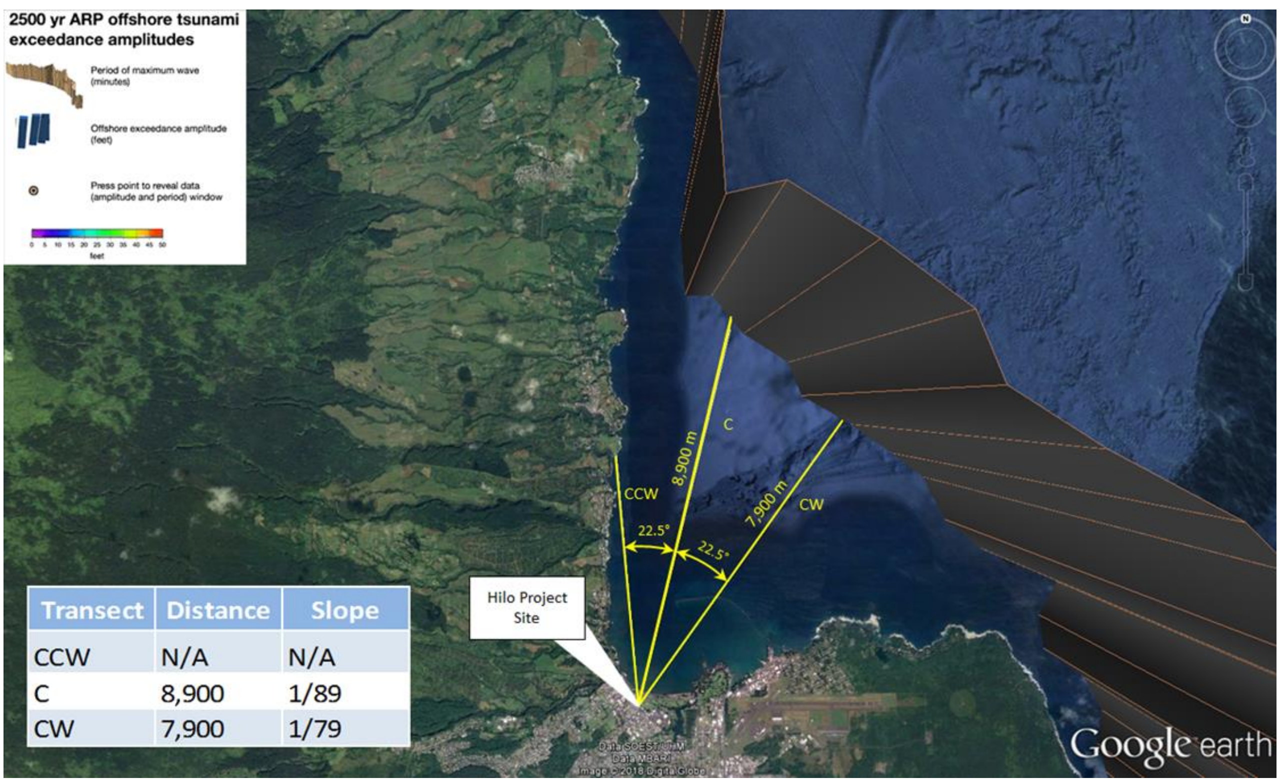

Figure C-5: Determination of average nearshore slope from 100 meter bathymetric line to shoreline along a line perpendicular to the shoreline and lines rotated 22.5 degrees to either side of the center line.

The average nearshore bathymetric slope along each transect is then computed using:

$$Slope = \frac{100}{distance} \quad \text{in meters}$$

or

$$Slope = \frac{328}{distance} \quad \text{in feet .}$$

The table in **Figure C-5** shows that the near shore slope is steeper than 1/100, therefore this project site would not create bores through prevailing nearshore bathymetric slope. However, the presence of an offshore fringing reef will cause the tsunami wave to break and form a bore.

Figure C-6: Historical photo of bore at Hilo from the 1946 Tsunami in Hilo, HI.

As seen in **Figure C-6**, there is photographic evidence of a bore being formed in Hilo during the 1946 Aleutian Tsunami and therefore bores must be considered.

C.5 Determination of Inundation Depth and Flow Velocity using EGLA

The Energy Grade Line Analysis (EGLA) is a stepwise procedure starting from the run up elevation at the mapped inundation limit, and working shoreward to get the flow parameters at the site of interest.

A spreadsheet was used to perform this operation along all three transects. The input values were the runup, including sea level rise referenced to MHW datum (**Table C-1** column 3), the inundation limit distance determined from the topographic profile (**Table C-1** column 5), a Manning's coefficient of 0.030

representing “all other cases” from ASCE 7 Table 6.6-1, and α = 1.3 representing bore conditions at the shoreline as specified in ASCE 7 Section 6.6.4. The resulting inundation depth profiles, both with and without the topographical elevation, are shown in **Figure C-7** and **Figure C-8** for the Center transect, **Figure C-9** and **Figure C-10** for the Counterclockwise transect, and **Figure C-11** and **Figure C-12** for the Clockwise transect.

The Clockwise transect results in the largest flow depth of 55.12 feet at the project site, which is the value of h_{max} that will be used in the subsequent design calculations.

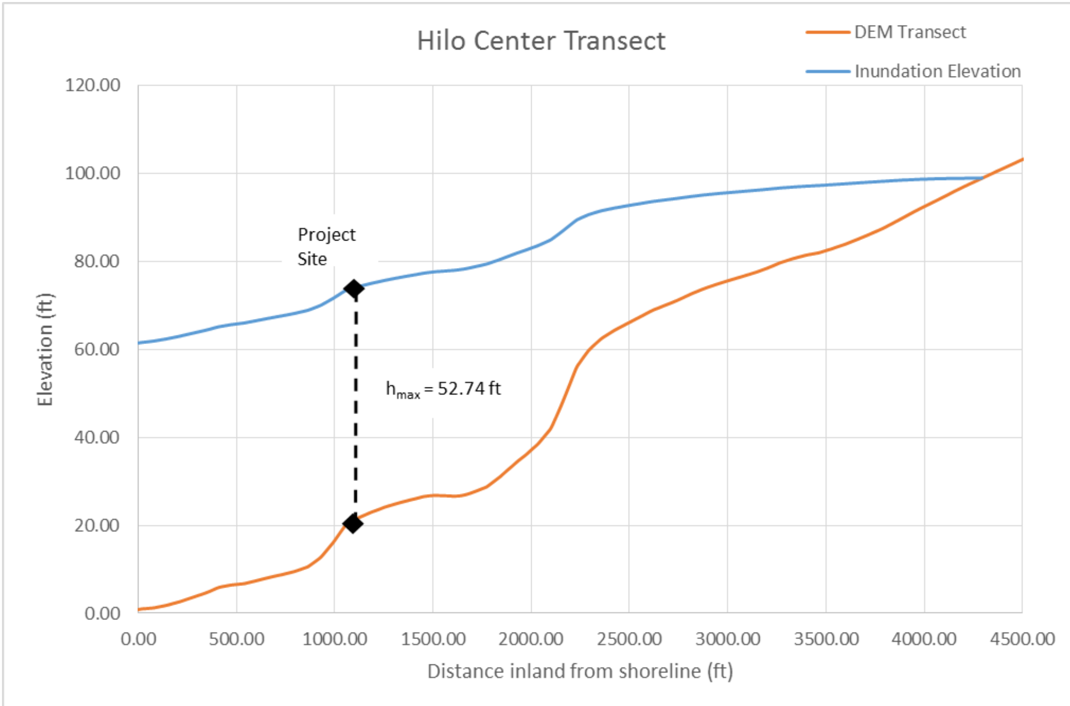

Figure C-7: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for center transect

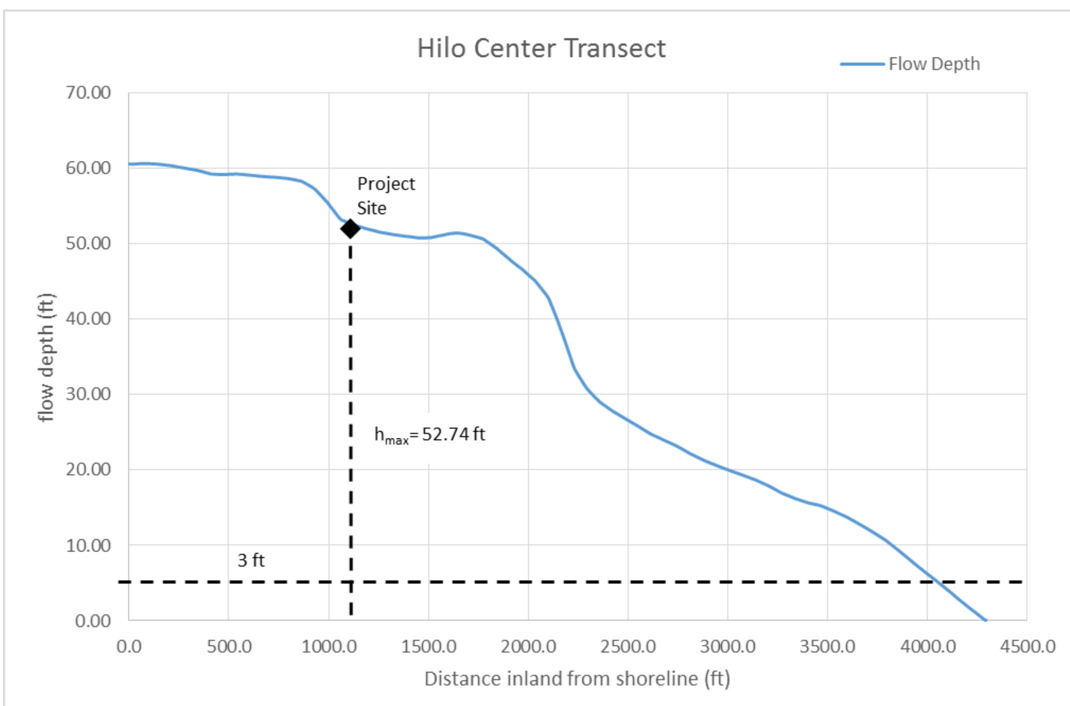

Figure C-8: Inundation depth (h_i) profile from Energy Grade Line analysis for center transect

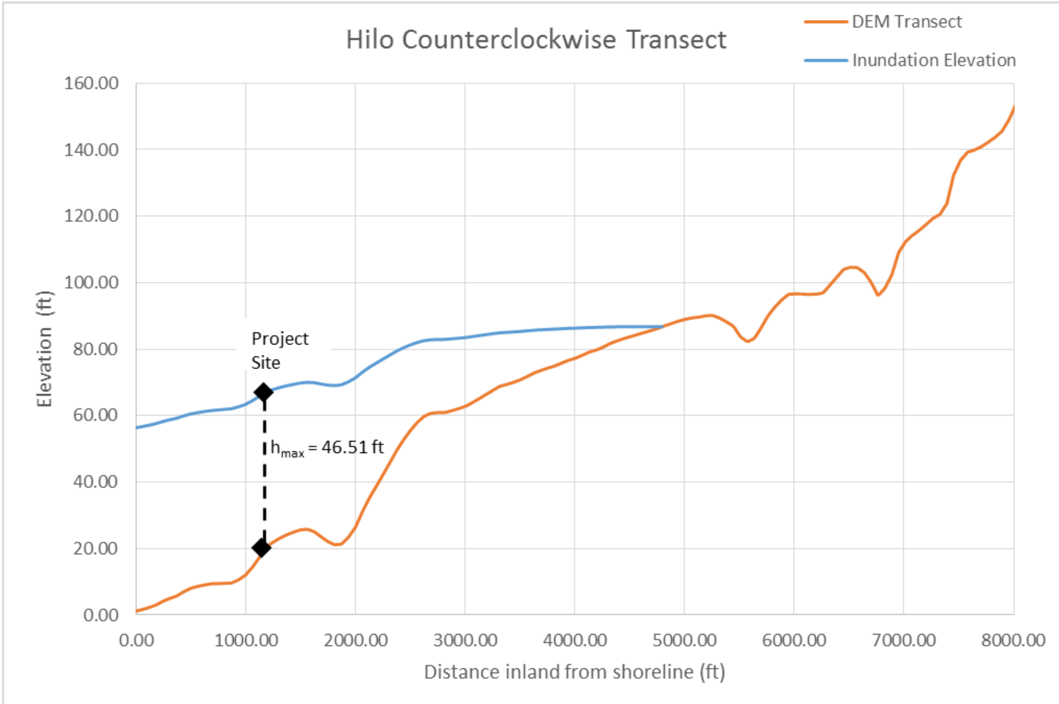

Figure C-9: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for counterclockwise transect

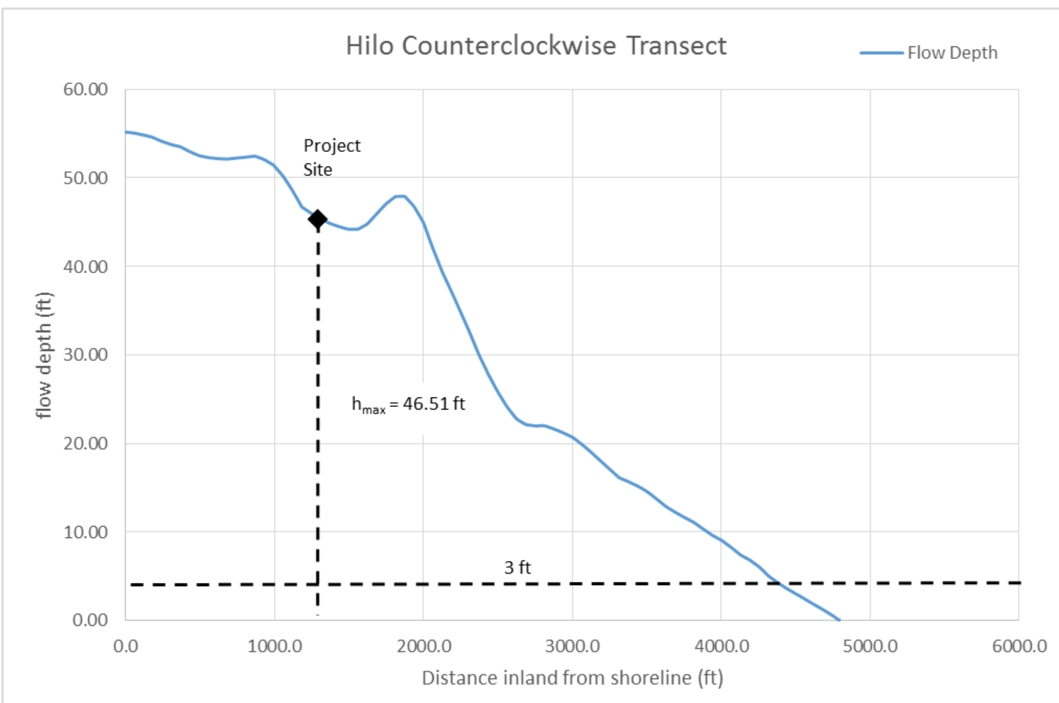

Figure C-10: Inundation depth (h_i) profile from Energy Grade Line analysis for counterclockwise transect

Figure C-11: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for clockwise transect

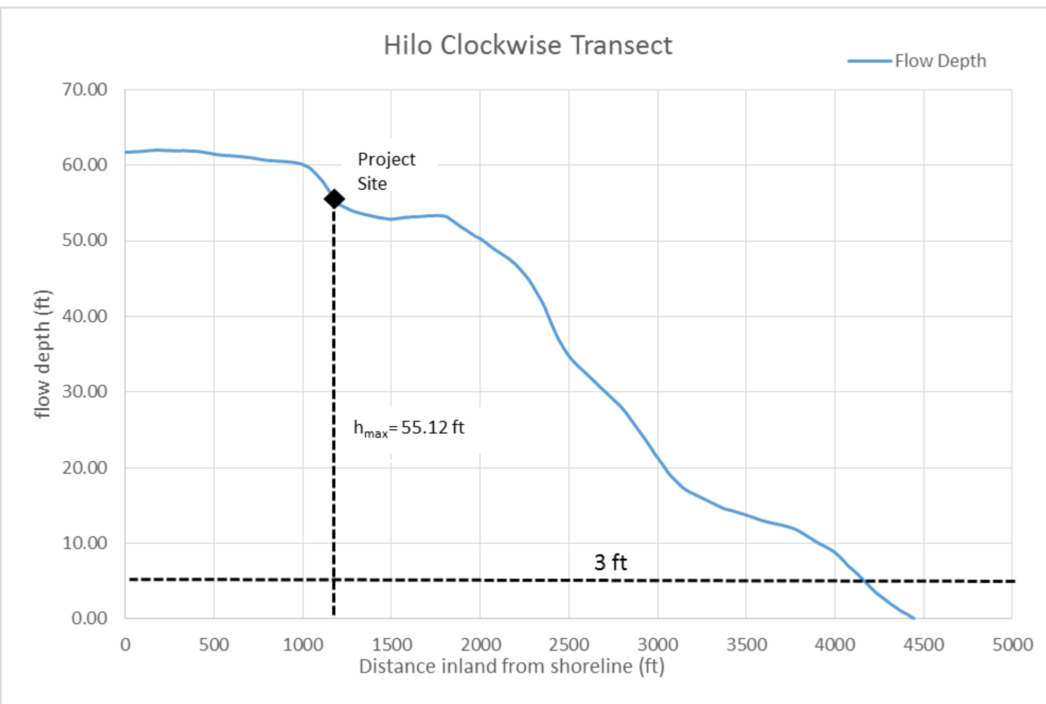

Figure C-12: Inundation depth (h_i) profile from Energy Grade Line analysis for clockwise transect

The flow velocity profiles across each transect as determined from the EGLA are shown in **Figure C-13**, **Figure C-14** and **Figure C-15** for the Center, Counterclockwise and Clockwise transects, respectively. The minimum flow velocity that may be considered is 10 ft/sec, which is indicated on each of the plots. As with the flow depth, the Clockwise transect produces the largest estimate of flow velocity at 46.88 ft/sec, which is the value of u_{max} that will be used in the design calculations.

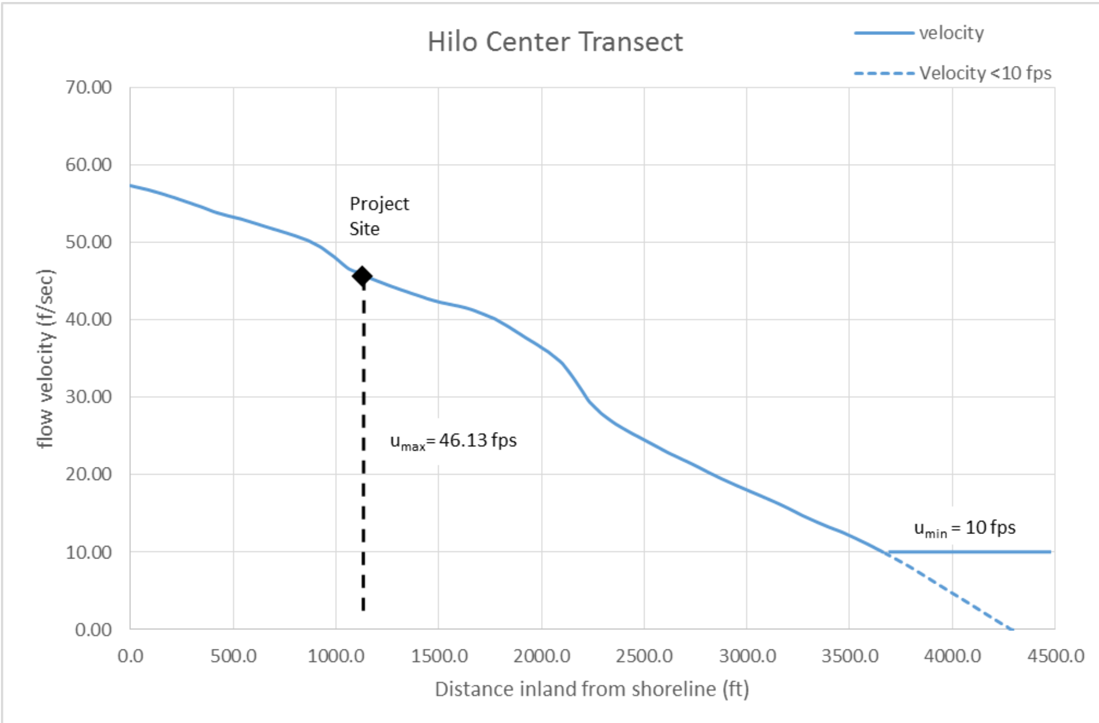

Figure C-13: Flow velocity (u_i) profile from Energy Grade Line analysis for Center transect

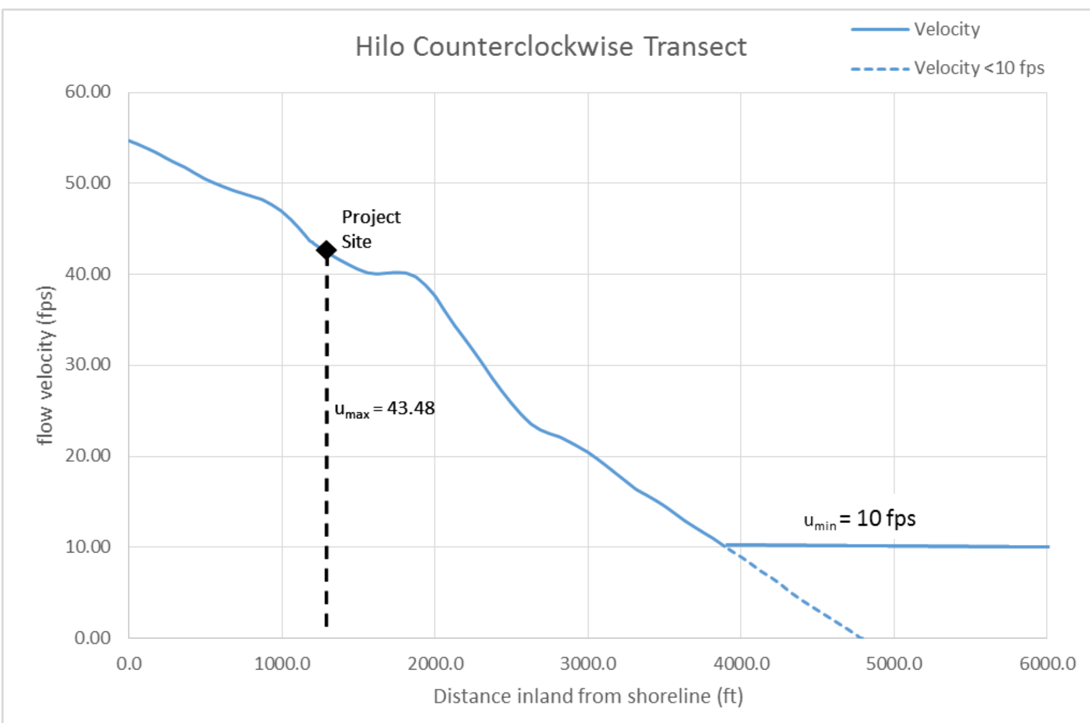

Figure C-14: Flow velocity (u_i) profile from Energy Grade Line analysis for Counterclockwise transect

Figure C-15: Flow velocity (u_i) profile from Energy Grade Line analysis for Clockwise transect

All of the flow depths and flow velocities determined from the EGLA are listed in **Table C-2**

Table C-2: Results of Energy Grade Line Analysis for three transects through Monterey project site.

Transect	Maximum Flow Depth, h_{max} (ft)	Maximum Flow Velocity, u_{max} (ft/sec)
Center	52.74	46.13
Counterclockwise	46.51	43.48
Clockwise	**55.12**	**46.88**

C.6 Prototype Concrete Buildings

C.6.1 6-Story Office Building

The 6-story office building consists of a Special Moment Resisting Frame on the perimeter and selected interior frames, and interior gravity columns supporting posttensioned floor slabs (See **Figure C-16**). The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: D

Response Spectrum Parameters: S_s = 1.5, S_1 = 0.6, S_{DS} = 1, S_{D1} = 0.6

Structural System Response Factors: R = 8, Ω_o = 3, C_d = 5.5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 74 ft. With a maximum flow depth of 55.12 ft, this building could function as a “Refuge of Last Resort” at the 6th level (62 ft) and roof (if accessible).

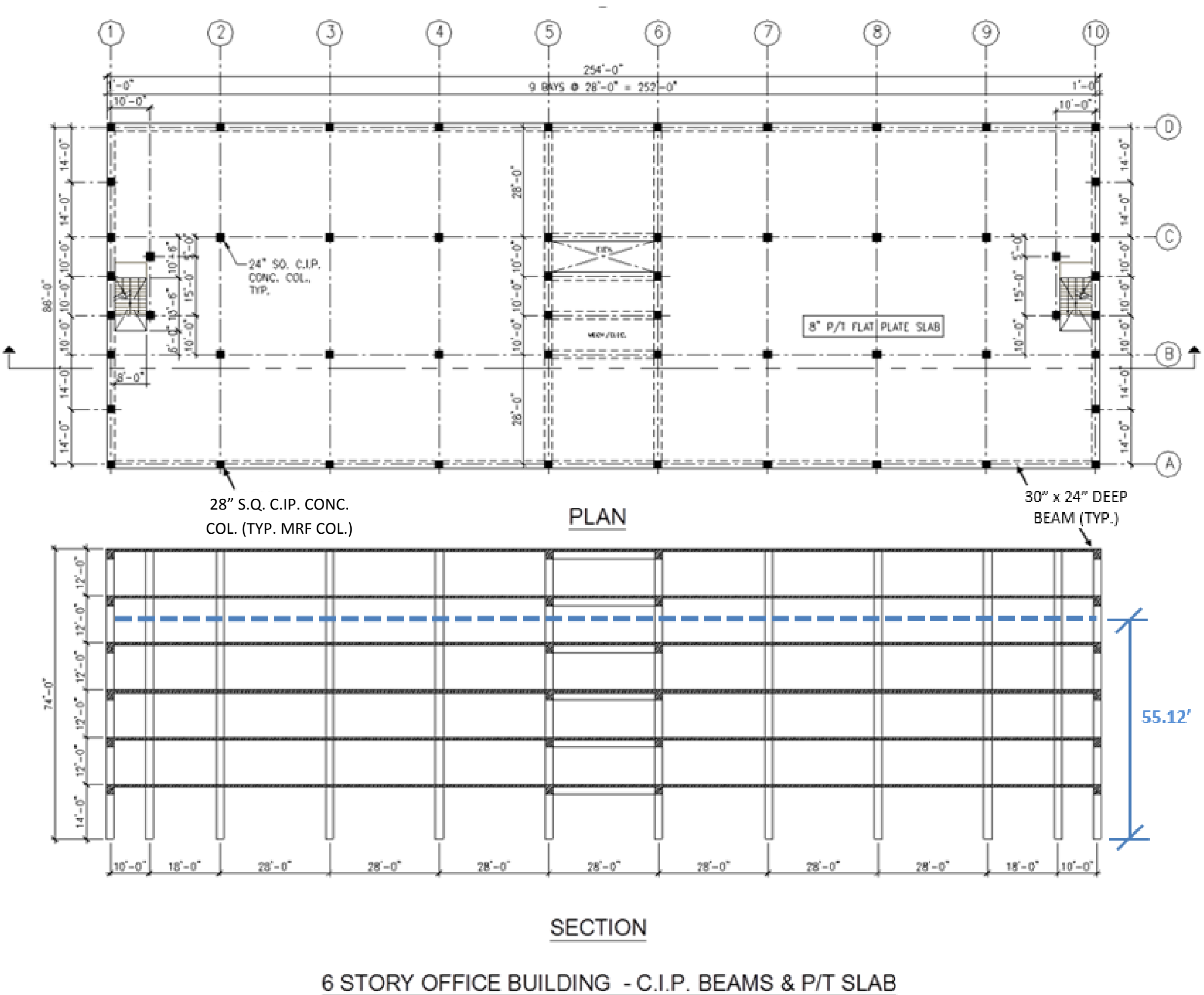

Figure C-16: 6-Story Office Building using Special Reinforced Concrete Moment Frames and posttensioned flat slab supported on gravity columns

C.6.2 7-Story Residential Building

The 7-story residential building consists of a Building Frame System with special reinforced concrete shear walls at exit stairs and elevator core, and interior gravity columns supporting posttensioned floor slabs (See **Figure C-17**). The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: D

Response Spectrum Parameters: S_s = 1.5, S_1 = 0.6, S_{DS} = 1, S_{D1} = 0.6

Structural System Response Factors: R = 6, Ω_o = 2.5, C_d = 5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 66 ft. With a maximum flow depth of 55.12 ft, this building could function as a “Refuge of Last Resort” at the 7th level (57 ft) and roof (if accessible).

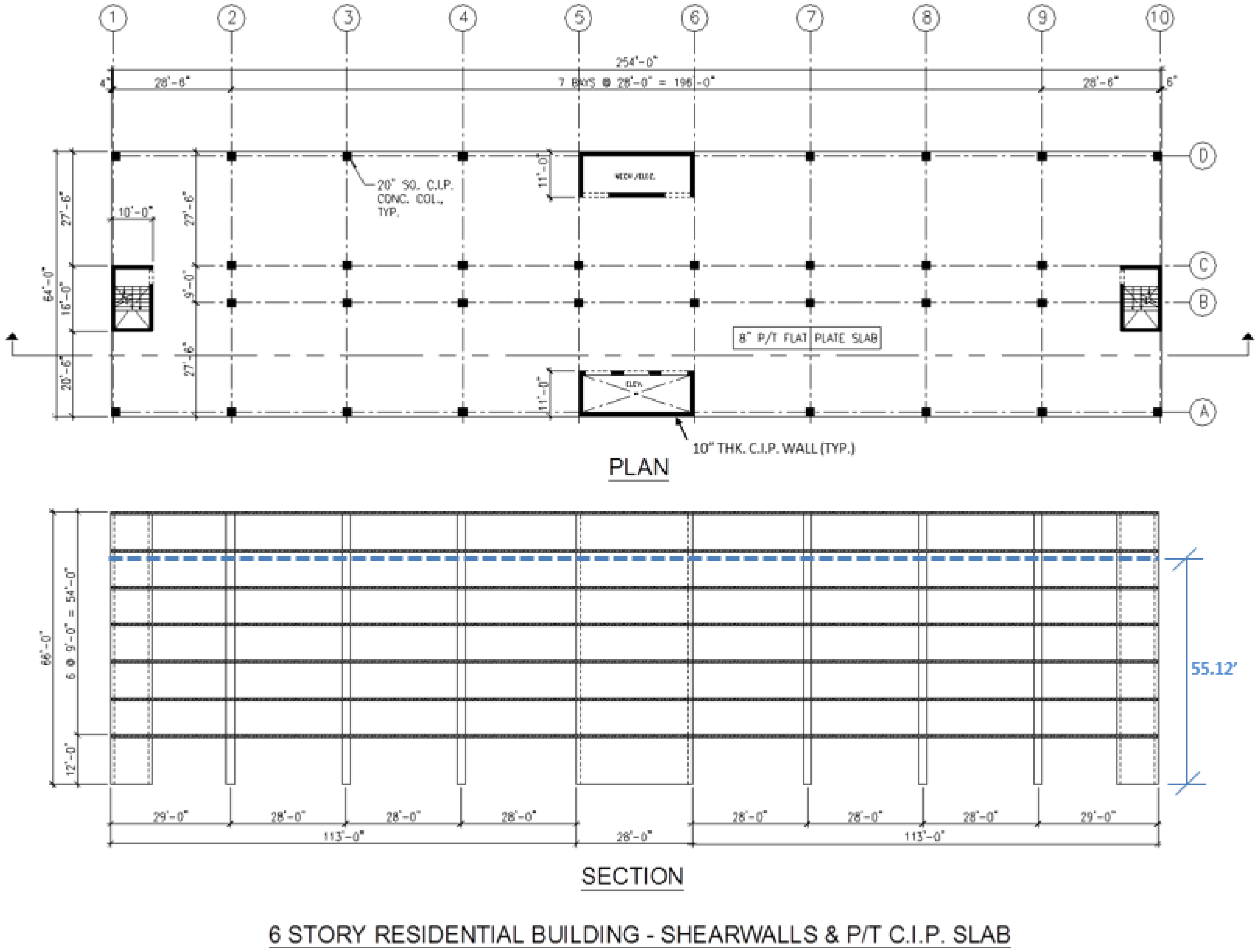

Figure C-17: 7 Story Residential Building using Special Reinforced Concrete Shear Walls and posttensioned flat slab supported on gravity columns

C.7 Tsunami Loading Summary

Table C-3 gives a summary of the tsunami loads determined for the two prototype buildings located at the selected site. The subsequent sections show the derivation of each of these values.

This example shows detailed calculation of the tsunami loads, along with evaluation of the structural system and components for these loads. Note that these calculations are far more detailed than would be necessary for a typical design project because the intent here is to provide a complete explanation of the various calculations and their application.

Table C-3: Summary of Tsunami Loading for Office and Residential Building

Flow Parameters	**Office Building**	**Residential Building**
Max. Inundation Depth, h_{max} (ft)	55.12	55.12
Max. Flow Velocity, u_{max} (fps)	46.88	46.88
Overall Building Lateral Loading (kips)		
Load Case 1	1,790	1,790
Load Case 2	19,744	19,744
Load Case 3	3,291	3,291
Component Loading (kips)		
Exterior Column Hydrodynamic Drag	3,482 [1]	3,482 [1]
Interior Column Hydrodynamic Drag	355.3	296
Exterior Column Debris Impact	107.25 [2]	107.25 [2]
Exterior Wall Debris Impact	-	107.25 [2]
Wall and Slab Loading (psf)		
Hydrodynamic Pressure on Walls	-	4,835
Stagnation Pressure in Mech/Elec Rm	-	2,418 [3]
Surge Uplift on Elevated Slabs	-	20

[1] Including effect of debris damming, C_{cx}, applied to column tributary width.

[2] Limited by log crushing capacity.

[3] Stagnation pressure acting outwards on structural walls and floor slab enclosing Mech/Elec room corresponding to the maximum velocity and corresponding flow depth.

C.8 Assumed Conditions

The following conditions are assumed to apply for this example:

13. The building is oriented with the longitudinal axis parallel to the shoreline.
14. The building has no basement.
15. The foundation system consists of deep piles with pile caps supporting all shear walls and all exterior columns. All pile caps are interconnected with grade beams.
16. The ground floor slab-on-grade has isolation joints at all columns, structural walls and grade beams.
17. The top of the first floor windows is 8 feet above grade, with the window sill at 3 ft.

18. The building location is not in the vicinity of a shipping container storage yard or port facility, and is therefore not subject to debris impact from shipping containers, ships or barges.
19. The non-structural exterior cladding spans vertically between floors.

C.9 Tsunami Design for Office Building

C.9.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

In lieu of performing detailed hydrostatic and hydrodynamic analysis, **Eqn. 6.10.1-1** provides a simplified but conservative estimate of the maximum lateral load on the building.

$$f_{uw} = 2.5 I_{tsu} \gamma_s h_{max}^2 = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 55.12^2 = 534.73\ kip/ft$$

Assuming $C_{cx} = 0.7$ controls per **Section 6.8.7**

Then $F = 0.7 \times 254 \times 534.73 = 95{,}074\ kips$

This lateral load can be compared with 0.75 Ω_o E_h = 0.75 x 3 x 2,435 = 5,479 kips < 95,074 kips. The detailed analysis for LC2 and LC3 must therefore be performed as shown below.

C.9.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

C.9.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} $\le h_{max}$ = 55.12 ft

$\le$ 14 ft (ground floor story height)

$\le$ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

For the sake of illustration, if we had assumed *that the ground floor consists of structural grade beams and integral slab on grade without isolation joints, and that the soil allowed ground water pressure increase below the building (ie. sandy or gravely subsoil), the buoyancy would need to be considered as follows:*

Section 6.9.1, Eqn. 6.9-1 $F_v = \gamma_s V_w$ = (1.1x64.0)(254' x 88' x 8')/1000 = 12,588 kips

Apply load combination: 0.9*D* + F_{TSU} + 1.2 H_{TSU}

where H_{TSU} = 0 *since scour is assumed uniform around the building perimeter*.

and building dead weight, D = 16,000 kips, *including foundation*.

Therefore net uplift = - 0.9 x 16,000 + 12,588 = -1812 kips, downward.

Overall uplift would therefore not be a concern, even if the ground floor were a structural slab capable of resisting the associated buoyancy pressures. This example also ignores any uplift resistance provided by the deep foundations.

In combination with buoyancy, Load Case 1 requires application of the associated hydrodynamic drag on the entire building.

Section 6.10.2, Eqn. 6.10-.2 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4575 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254' overall width of building

h = 8'

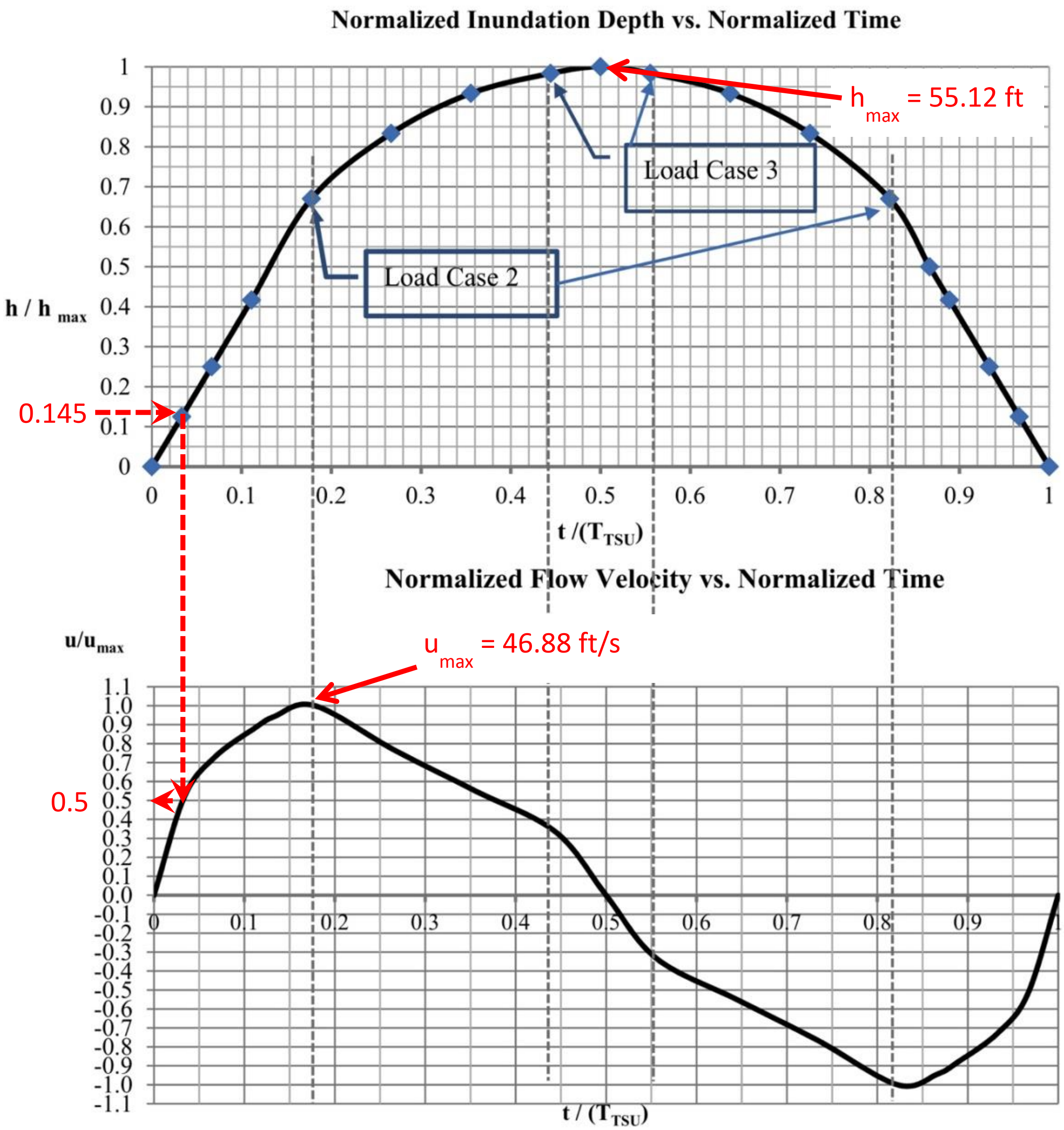

Figure C-18: Determining "u" for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/55.12 = 0.145. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.038. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.5. Therefore the flow velocity is u = 0.5 x 46.88 = 23.44 fps.

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 23.44^2)/1000 = 1{,}790\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure at the

first floor level would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail so as to relieve lateral load on the structural frame. Note that only portion of this load will go to the second floor slab, which therefore has to be resisted by the lateral force resisting system. The majority of the load will go directly to the grade beam/foundation system. The entire lateral load must be resisted by the deep foundation assuming maximum scour has already occurred.

C.9.2.2 Load Case 2: Maximum Flow Velocity

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is $2/3h_{max}$ = 2/3 x 55.12 = 36.75 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/36.75 = 6.91)

Since the inundation depth of 36.75 feet exceeds the bottom of the fourth floor beams (14'+12'+12 – 24") = 36', the inundated area of the beams must be included in the closure coefficient, which is given by:

h_{sx} = 36.75 ft.

$h_{col\ EQ}$ = 36.75'-24"-24"-.75" = 32' (Clear height of submerged Moment Resisting Frame columns)

$A_{col\ EQ}$= 32'x2.33'x40 = 2,982 ft^2 (40 MRF earthquake columns each 28" wide)

$h_{col\ Grv}$= 36.75'-8"-8"= 35.41' (clear height of submerged gravity load columns)

$A_{col\ Grv}$= 35.41'x2'x16 = 1,133 ft^2 (16 gravity load column, each 2' wide)

A_{wall} = 0 ft^2 (no walls in MRF structure)

A_{beam} = 24"x254'x2.375/12= 1,206 ft^2 (2x24" deep beam goes 0.75' of 4th level beam)

$$C_{cx} = \frac{\sum(A_{col} + A_{wall}) + 1.5A_{beam}}{Bh_{sx}} = \frac{\sum((1133 + 2982) + 0) + 1.5 \times 1206}{254' \times 36.75'} = 0.635 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

$u = u_{max}$ = 46.88 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(36.75 \times 46.88^2)/1000 = 19{,}744\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal or inland elevation of the building over a height of 36.75 feet above grade as shown in **Figure C-19**. The lateral force resisting system for the structure at the first, second and third floor levels would be evaluated for this load.

C.9.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 55.12 ft. and the flow velocity is 1/3u_{max} = 1/3 x 46.88 = 15.63 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/55.12 = 4.6)

Since the inundation depth of 55.12 feet exceeds the bottom of the fifth floor beams, (14'+12'+12+12' – 24") = 48', the inundated area of the second through fourth floor beams must be included in the closure coefficient, which is given by:

h_{sx} = 55.12 ft.

$h_{col\ EQ}$ = 55.12'-4x24"= 47.12'

$A_{col\ EQ}$= 47.12'x2.33'x40 = 4,397 ft^2

$h_{col\ Grv}$= 55.12'-4x8"= 52.45'

$A_{col\ Grv}$= 55.12'x2'x16 = 1,678 ft^2

A_{wall} = 0 ft^2

A_{beam} = 24"x254'x4= 2032 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum((4397+1678)+0)+1.5\times 2032}{254\prime\times 55.12\prime} = 0.652 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 15.63 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(55.12 \times 15.63^2)/1000 = 3{,}291\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 55.12 feet above grade as shown in **Figure C-20**. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth for hydrodynamic load and debris impact.

C.9.3 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category D, **Section 6.8.3.4** permits the use of $0.75\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear. From the seismic design of this structure, E_h = 2,435 kips. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 3 \times 2{,}435 = 5{,}479\ kips$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 19,744 kips applied over a height of 36.75 ft. A portion of this load will be resisted by the grade beam/foundation system as shown in **Figure C-19**, reducing the overall load by 3,761 kips. Therefore, V_{TSU} = 19,744 – 3,761 = 15,983 kips. Applying the LFRS assessment gives:

$$0.75\Omega_o E_h = 5{,}479\ kips < 15{,}983\ kips \quad \therefore Not\ OK$$

So the lateral force resisting system does not have the capacity to resist the overall tsunami loads. The seismic base shear must be increased so that this LFRS check is met. The building must therefore be designed for a seismic base shear, E_h, of:

$$E_h = \frac{V_{TSU}}{0.75\Omega_o} = \frac{15{,}983}{0.75 \times 3} = 7{,}104\ kips$$

This seismic base shear must be distributed up the height of the building following ASCE 7 seismic design provisions. The ETABS model used for the original wind and seismic analysis of the building was used for this analysis, resulting in the column forces shown in **Figure C-22** to **Figure C-26** for floors one through five, respectively.

While acting as part of the lateral force resisting system, these columns are also subjected to component drag or debris impact loads. According to ASCE 7 Section 6.8.3.5, the columns in the inundated floors must be designed and detailed for these higher forces "*that result from the overall tsunami forces on the structural system combined with any resultant actions caused by the tsunami pressures acting locally on the individual structural components for that direction of flow*". All members of the LFRS must resist the forces resulting from the overall system analysis, in combination with hydrodynamic and impact loads acting on the member itself.

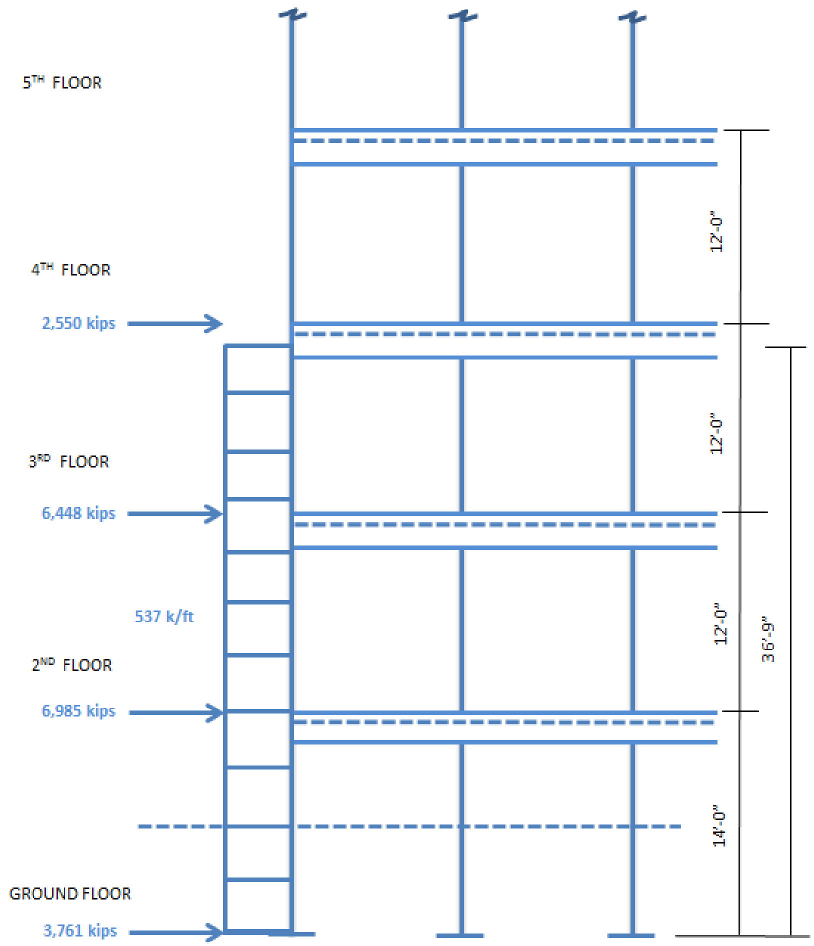

Figure C-19: LC2 Tsunami loads on overall Hilo office building

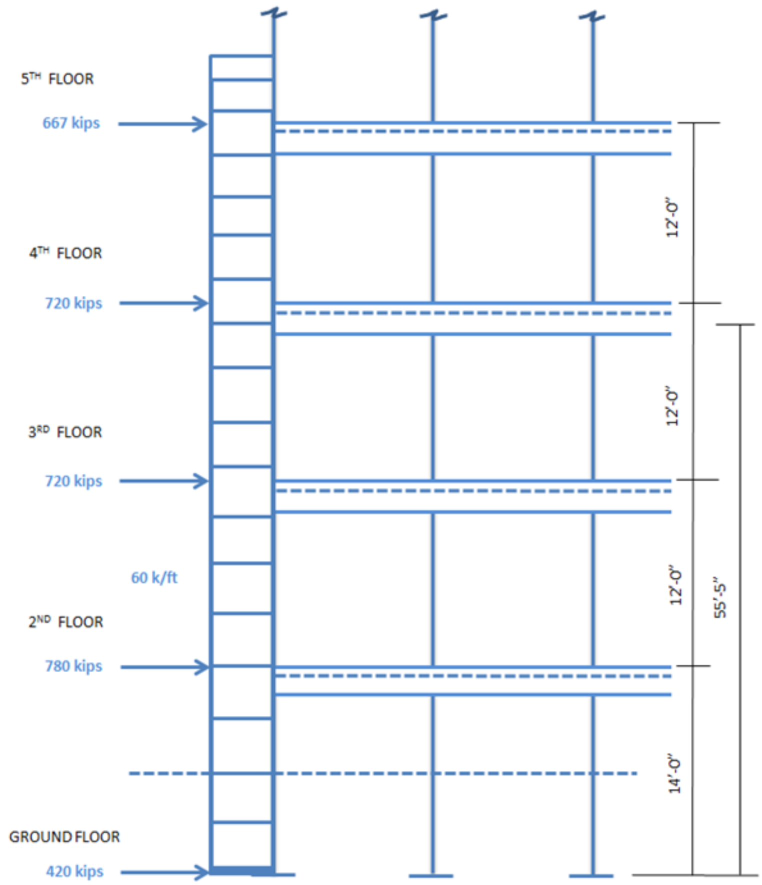

Figure C-20: LC3 Tsunami loads on overall Hilo office building

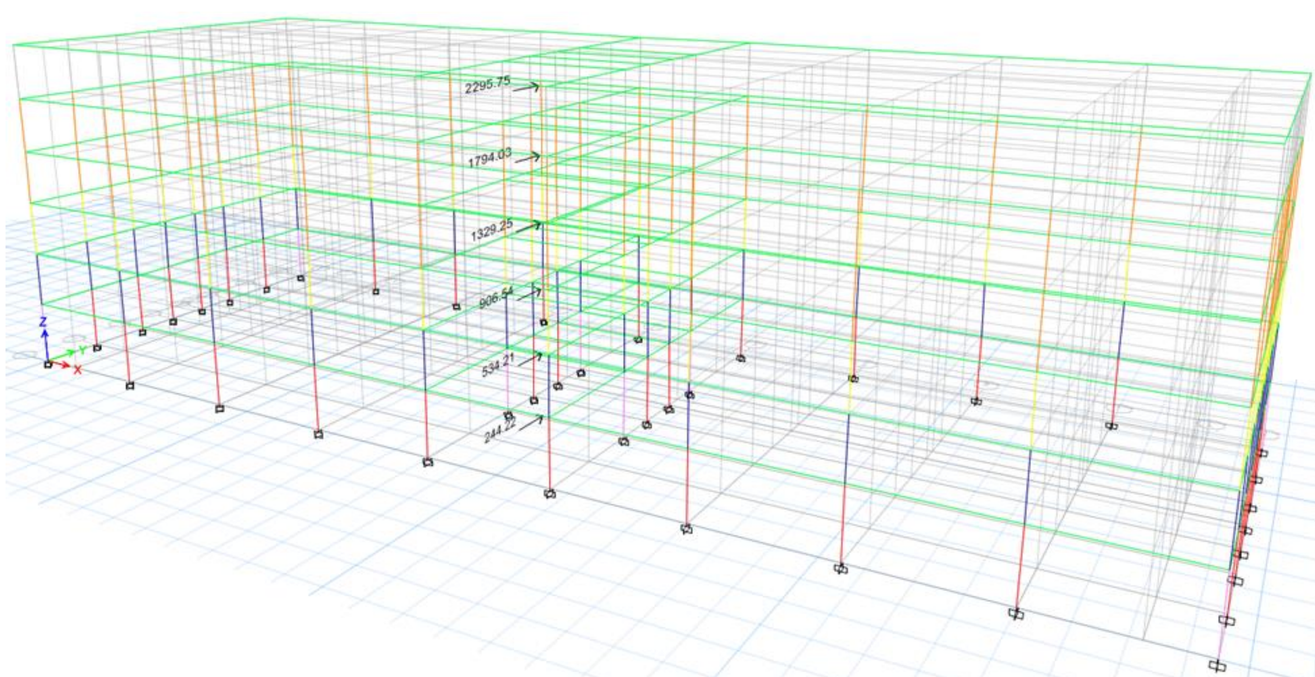

Figure C-21: ETABS computer model of Moment Resisting Frame office building (with seismic lateral loads shown).

Floor 1

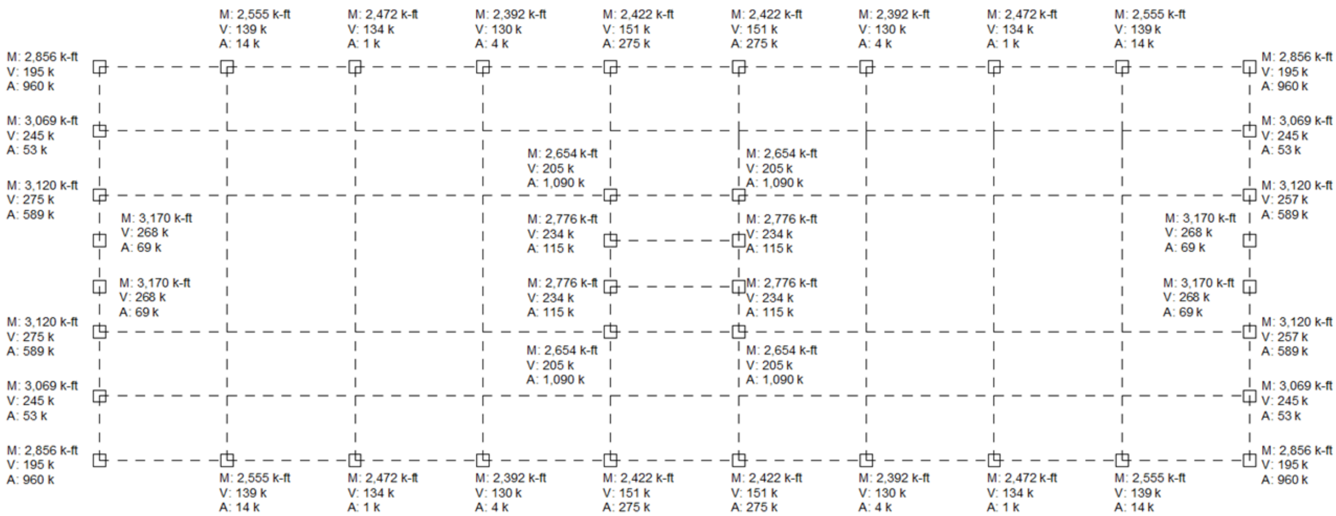

Figure C-22: Maximum forces in the first floor columns due to increased seismic base shear

Floor 2

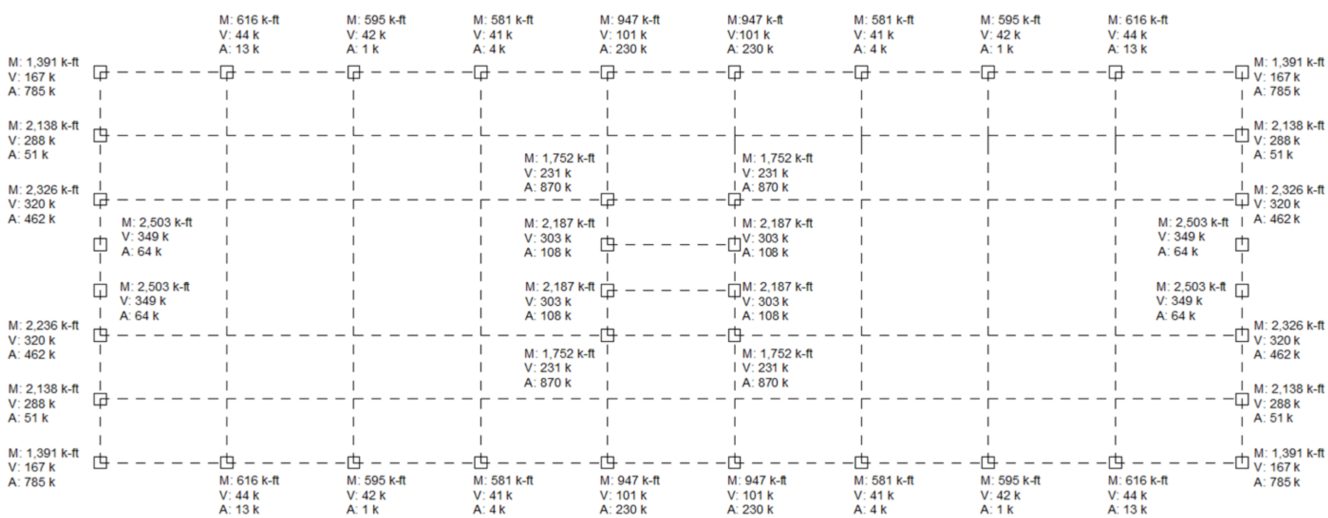

Figure C-23: Maximum forces in the second floor columns due to increased seismic base shear

Floor 3

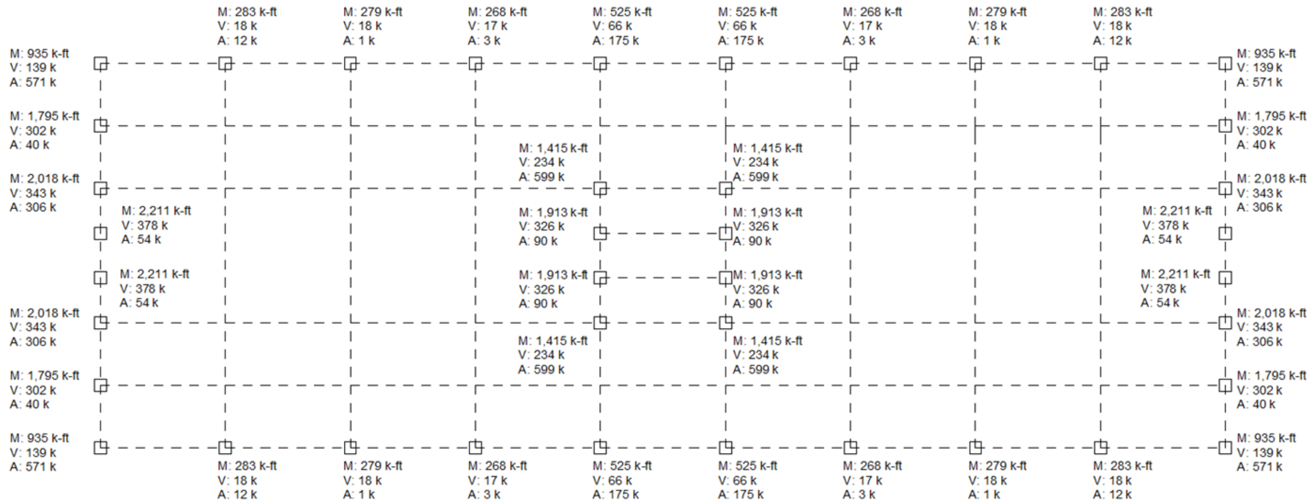

Figure C-24: Maximum forces in the third floor columns due to increased seismic base shear

Floor 4

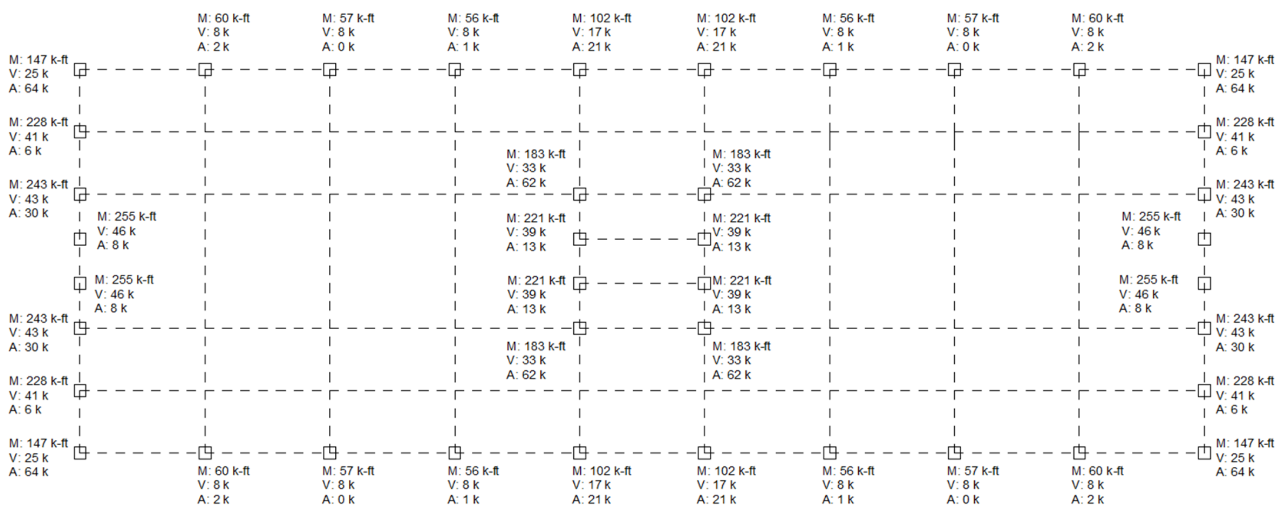

Figure C-25: Maximum forces in the fourth floor columns due to increased seismic base shear

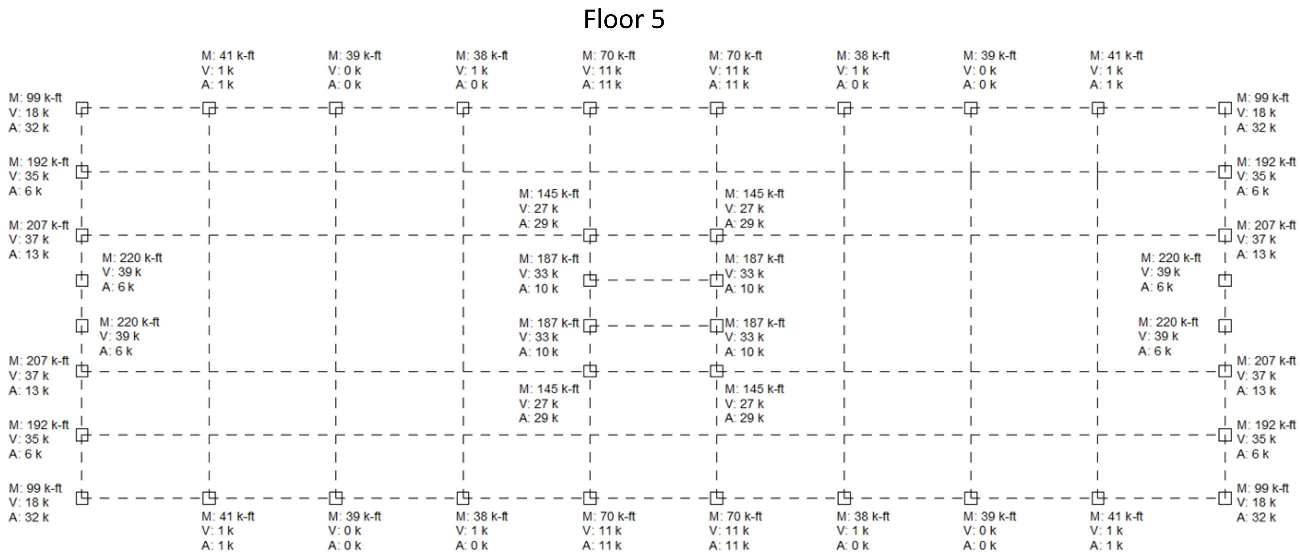

Figure C-26: Maximum forces in the fifth floor columns due to increased seismic base shear

C.10 Component Design

C.10.1 Drag Force on Components - Section 6.10.2.2

C.10.1.1 Exterior Columns

For Load Case 1, the exterior cladding is assumed to remain intact. Since the cladding spans vertically between floors for this example building, none of the hydrodynamic lateral load in LC1 will be applied directly to the ground floor columns. [*Note that if the exterior cladding were supported by girts which transferred lateral load to the columns, then the columns would need to be designed for this load*.]

For Load Cases 2 and 3, the exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** will require that C_d = 2.0 and the width dimension, b, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Previous calculation of C_{cx} showed that the default value of 0.7 controls for LC2 and LC3 for this building, Therefore b = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 36.75 ft and u_{max} = 46.88 fps.

The hydrodynamic drag is computed using **Eqn 6.10-4** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(36.75 \times 46.88^2)/1000 = 3{,}482\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 3,482/36.75 = 94.77 kips/ft over the lower 36.75 feet of the column. The column must be designed for this load combined with gravity loads using the load combinations in **Section 6.8.3.3**. In addition, because the exterior columns are part of the LFRS, these component loads must be combined with the systemic forces and the column designed for the combined loads.

C.10.1.2 Interior Columns

Interior columns are 24" (2 ft) square R.C. columns. For Load Case 1, the interior is not yet inundated, so there are no hydrodynamic loads on the interior columns. The controlling load case will be LC2, when the inundation depth is h_e = 36.75 ft and u_{max} = 46.88 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**)

Therefore $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 2.0(36.75 \times 46.88^2)/1000 =$ $355\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 355/36.75 = 9.6 kips/ft over the lower 36.75 feet of the column. This load must be combined with gravity loads using the load combinations in **Section 6.8.3.3** and the column capacity verified.

C.10.2 Other Hydrodynamic Loads

No other hydrodynamic load conditions apply to this building since there are no structural walls and the spandrel beam is integral with the slab so the lateral load on the beam will transfers directly to the slab diaphragm.

C.10.3 Debris Impact Loads - Section 6.11

The inundation depth at the site exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads per **Section 6.11**.

C.10.3.1 Detailed Debris Impact Calculation for Office Building

Wood Logs and Poles - Section 6.11.2

The nominal maximum instantaneous debris impact force is given by **Eqn. 6.11-2** as:

$$F_{ni} = u_{max}\sqrt{km_d}$$

Where u_{max} = 46.88 fps

$k = EA/L$ for the wood log with a minimum value of 350 k/in ($4.2\text{x}10^6$ lb/ft)

m_d = 1000/32.2 = 31.1 slugs for the minimum 1000 lb log.

Therefore: $F_{ni} = u_{max}\sqrt{km_d} = 46.88\sqrt{4.2 \times 10^6 \times 31.1}/1000 = 535\ kips$

The design instantaneous debris impact force is then given by **Eqn. 6.11-3** as:

$$F_i = I_{tsu} C_0 F_{ni} = 1.0\ \times 0.65 \times 535 = 348\ kips$$

The impulse duration is given by **Eqn. 6.11-4** as:

$$t_d = \frac{2m_d u_{max}}{F_{ni}} = \frac{2\times31.1\times46.88}{535{,}407} = 0.0054\ sec$$

The column can be designed using a dynamic analysis by applying an impulsive rectangular pulse with magnitude F_i and duration t_d. Alternatively an equivalent elastic static analysis can be performed of the column subjected to F_i multiplied by a dynamic response factor, R_{max}, given in **Table 6.11-1**. The ratio of impact duration to natural period of the impacted structural element is obtained using t_d and the natural period of the column assumed to be fixed-fixed. For this case, the natural period is given by;

$$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}}$$

Where L = unbraced column length = 14' – 24" = 12 ft for the ground floor columns.

ρ = column mass per unit length = 2.333'x2.333'x150pcf/32.2ft/s^2 = 25.36 slugs/ft

E = modulus of elasticity of the column concrete = 3600 ksi = 518.4x10^6 psf

I = moment of inertia of column section = $bd^3/12$ = 2.333x2.333^3/12 = 2.47 ft^4

Therefore $$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}} = 2\pi\left[\frac{12.17^2}{22.373}\right]\sqrt{\frac{25.36}{518.4\times10^6\times2.47}} = 0.00444\ sec$$

The ratio of impact duration to column natural period is therefore t_d/T_{col} = 0.0054/0.00444 = 1.22.

Table 6.11-1 gives the dynamic response factor R_{max} = 1.6, therefore the equivalent static load is given by;

$$F_{es} = R_{max}F_i = 1.6 \times 348 = 557\ kips.$$

This exceeds the maximum required impact force of 107.25 kips (See 1.10.5.1 below), therefore the column can be evaluated for a lateral point load of 107.25 kips applied at locations which are critical for flexure and shear.

C.10.4 Impact by Vehicles – Section 6.11.3

The impact force is given as $F_i = I_{tsu}$ x 30 = 30 kips. This will not control over the log impact load determined above.

C.10.5 Impact by Submerged Tumbling Boulder and Concrete Debris – Section 6.11.4

Because h_{max} = 55.12 ft > 6 ft, an impact force of $F_i = I_{tsu}$ x 8 = 8 kips shall be applied at 2ft above grade. This will not control over the log impact load determined above.

C.10.5.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330C_0 I_{tsu} = 330 \times .65 \times 1.0 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force will be reducible by 50%, or 107.25 kips. This load must be applied to the exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with hydrodynamic loads on the column, but it must be combined with systemic loads if the member is part of the lateral force resisting system. In the event that this load exceeds the column capacity, a detailed debris impact analysis can be performed. Debris impact loads are not applied to interior columns.

C.11 Column Design for Tsunami Loads

C.11.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 from **Figure C-16**. The column is part of the lateral force resisting system for longitudinal seismic load designed and detailed for Seismic Design Category D. The column has been designed for gravity and seismic loads resulting in the cross-section shown in **Figure C-27** and **Figure C-28** at the ground floor level and **Figure C-29** and **Figure C-30** for the remaining floor levels. The column will now be checked for tsunami load combinations.

Seismic design of the columns requires additional column ties to ensure ductility of the yield zones at each end of the column. These zones have a length equal to the maximum column cross-section dimension, in this case 28 inches. The critical shear force in this yielding zone occurs at a distance "*d*" from the top and bottom of the column, where *d* = 28 - 1.5 - 0.5 - 0.635 = 25.365 in. The critical shear force for the internal section of the column occurs at "d + h" from the edge of the column, where d + h = 25.365 + 28 = 53.365 in. The column ties required for seismic design will be evaluated for the shears induced by the tsunami both in the end section and center section of the column (**Figure C-31**).

Floor 1

End Section (A)

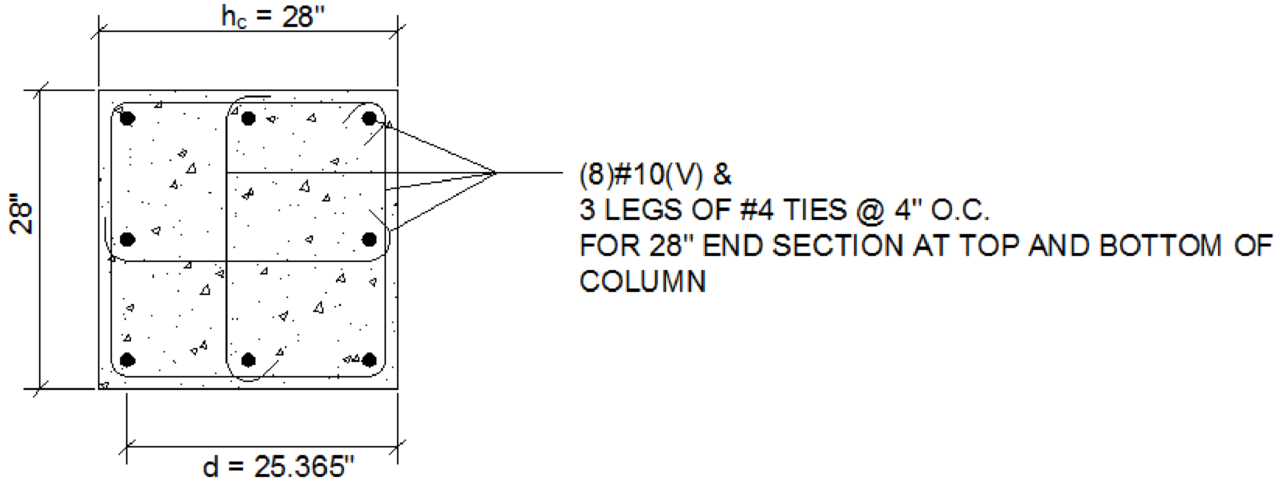

Figure C-27: Exterior column, cross-section at end of column at ground floor level based on SDC D design.

Center Section (B)

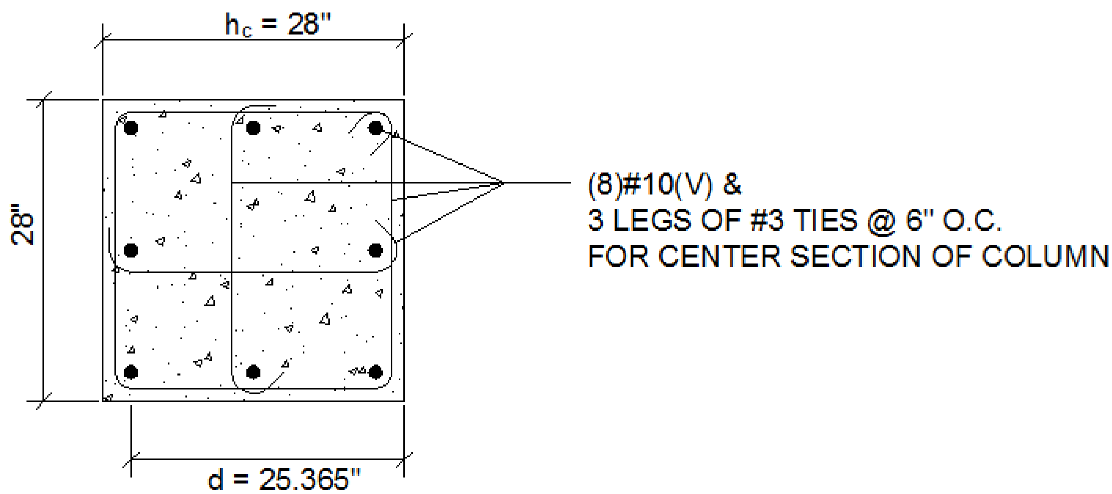

Figure C-28: Exterior column, cross-section at center of column at ground floor level based on SDC D design.

Floor 2 – 6

End Section (A)

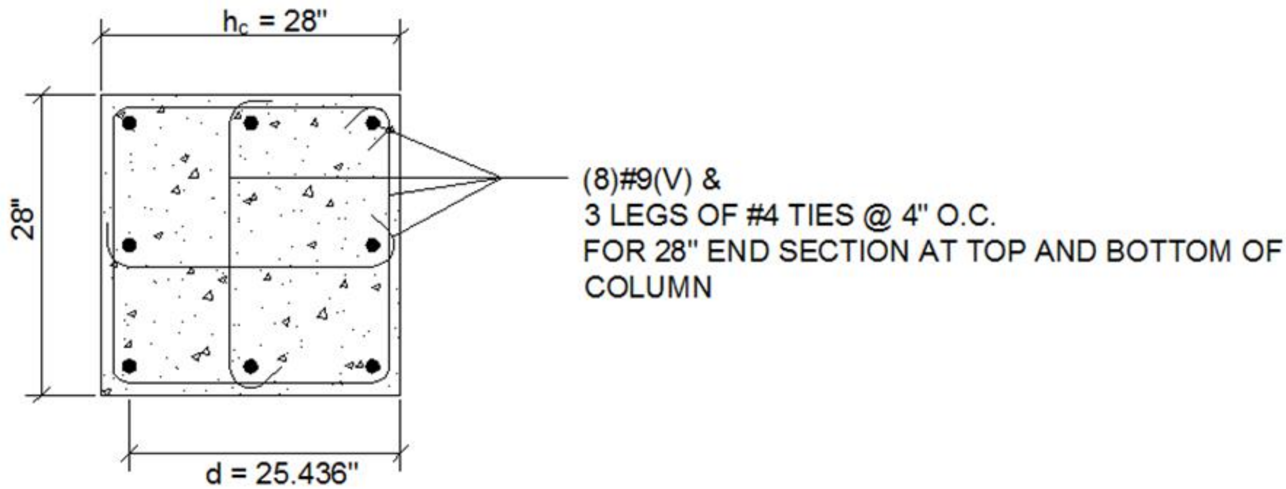

Figure C-29: Exterior column, cross-section at end of column at 2nd – 6th floor levels based on SDC D design.

Center Section (B)

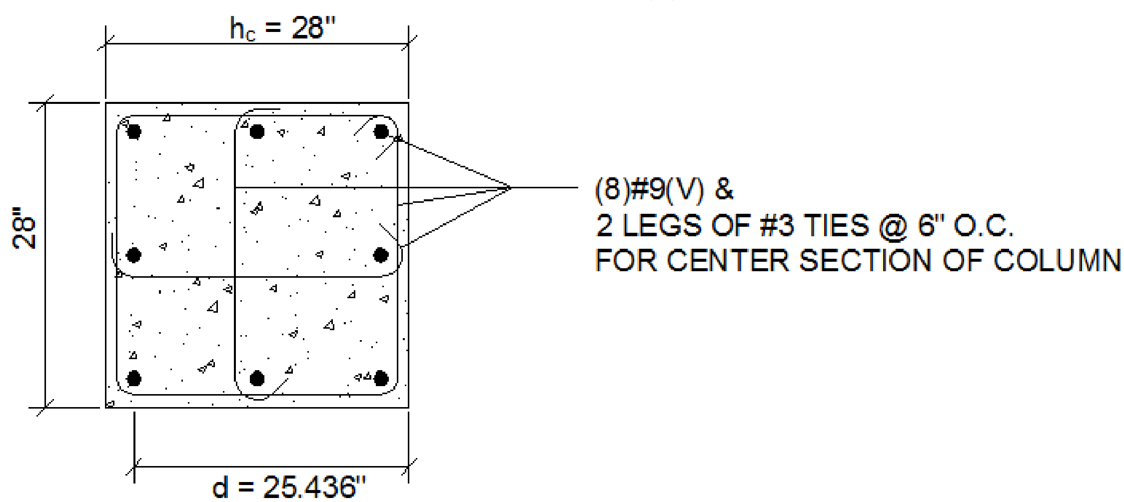

Figure C-30: Exterior column, cross-section at center of column at 2nd – 6th floor levels based on SDC D design.

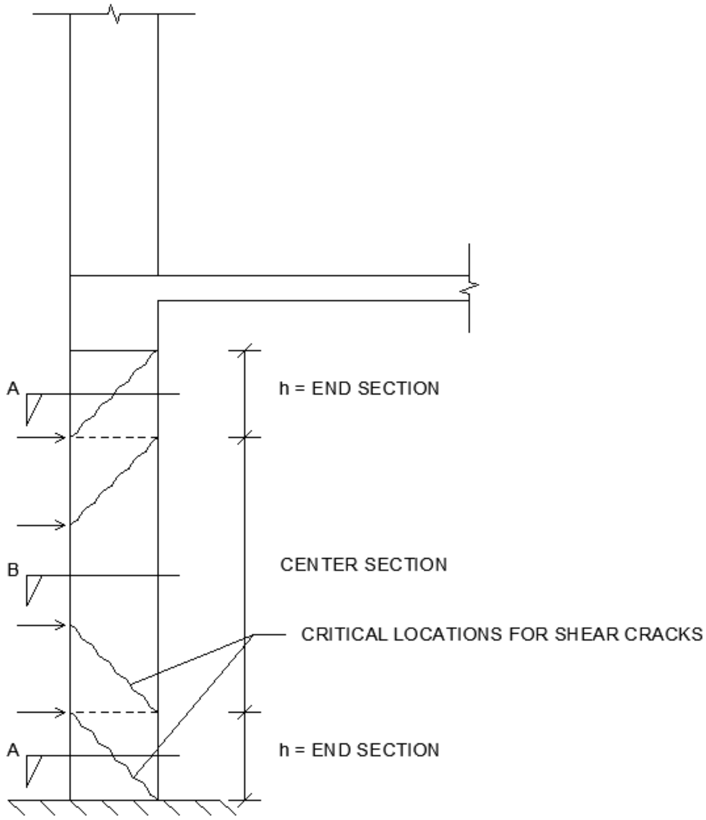

Figure C-31: Typical exterior column elevation showing end and center sections

C.11.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 15 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

$$P_D = [120(28)(15)(6)+(1.16)(2.5)(150)(28-2.5)+90(28)(5)+2.5^2(150)(74)]/1000 = 395\ k$$

Floor Live load reduction factor $= 0.25+15/[4(15)(28)(5)]^{0.5} = 0.414$,

therefore, live load at the column base is: $P_L = 0.414[65(15)(28)(5)]/1000 = 56.5\ k$

Roof Live Load reduction factor $= R_1R_2= [1.2-(0.001)(15)(28)](1.0) = 0.78$,

therefore, column roof live load is: $P_{Lr} = 0.78(20)(15)(28)/1000 = 6.55k$

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

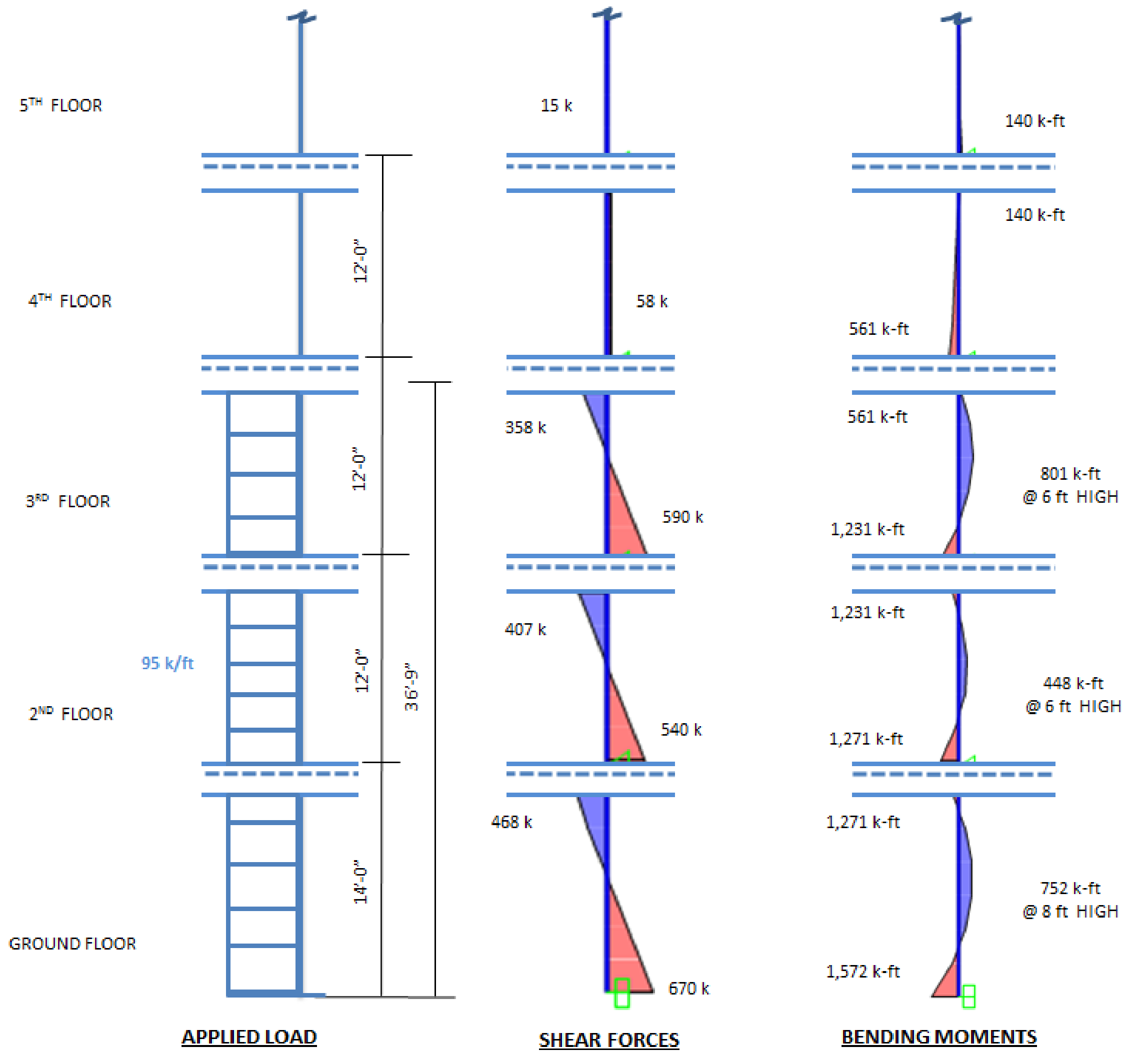

Figure C-32: Hydrodynamic loading on exterior column of the Hilo office building due to Load Case 2

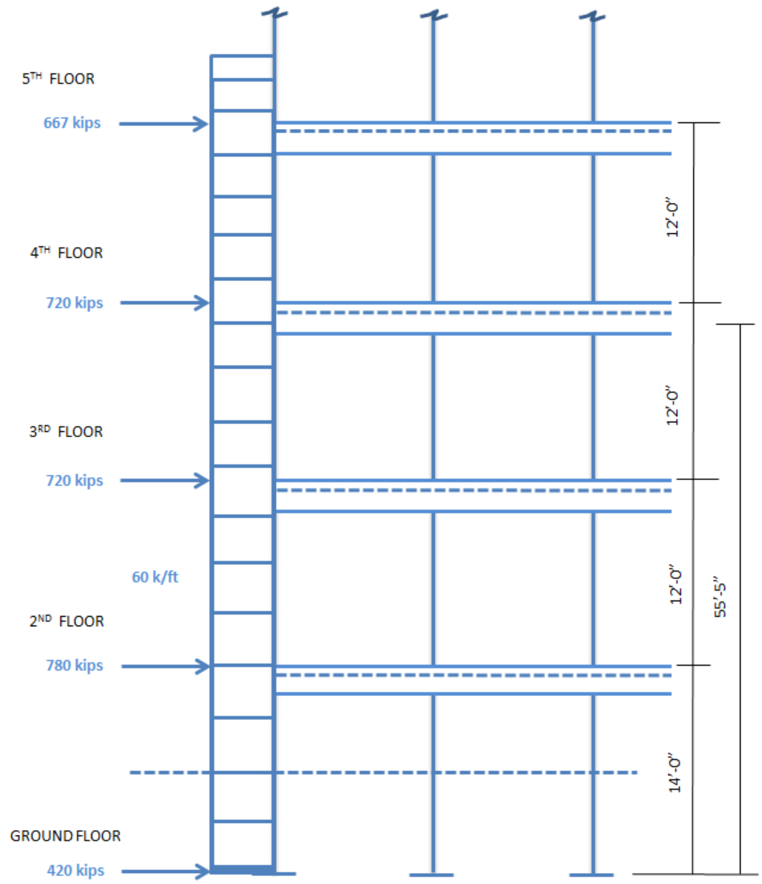

Figure C-33: Hydrodynamic loading on exterior column of the Hilo office building due to Load Case 3

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just below the beam at each inundated floor for the maximum shear in the end section of the column. A log strike is also assumed to act just outside the end section (at "d + h_c") and at the mid-height of the clear column height for the maximum shear force and bending moment in the center section, respectively. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure C-34** to **Figure C-38**. The resulting shear force and bending moment diagrams for log impact at a distance "d + h_c" from the end of the column at each floor level are shown in **Figure C-39** to **Figure C-43**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown in **Figure C-44** to **Figure C-48**.

Impact load at d:

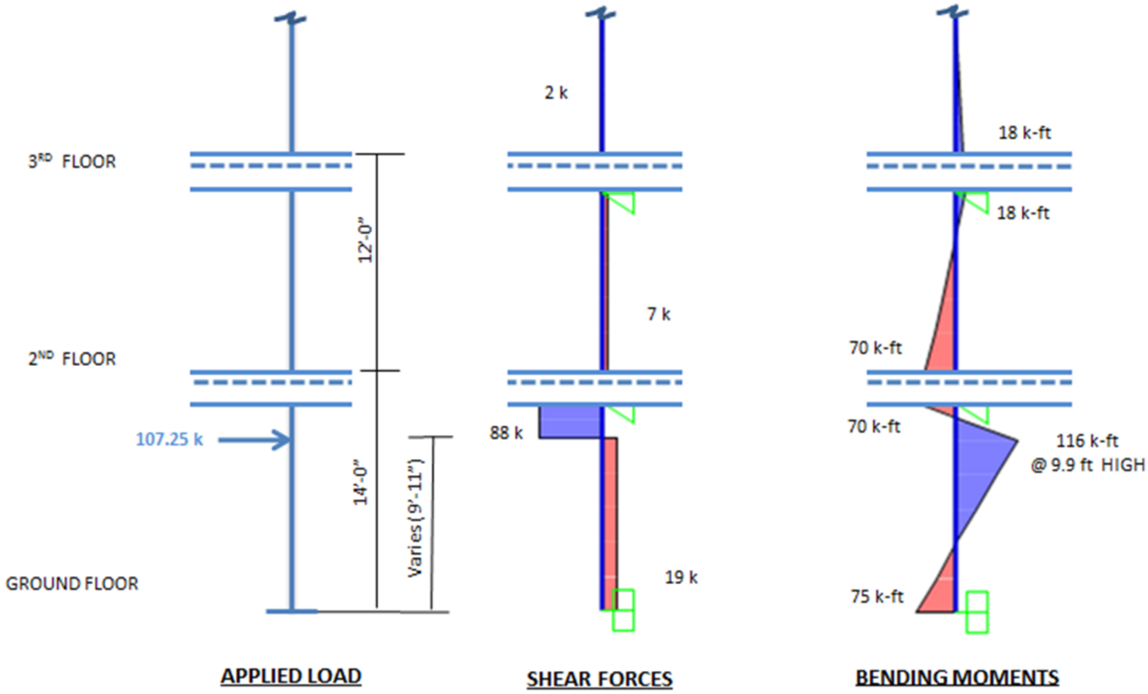

Figure C-34: Impact load applied at "d" away from the end of column on the ground floor

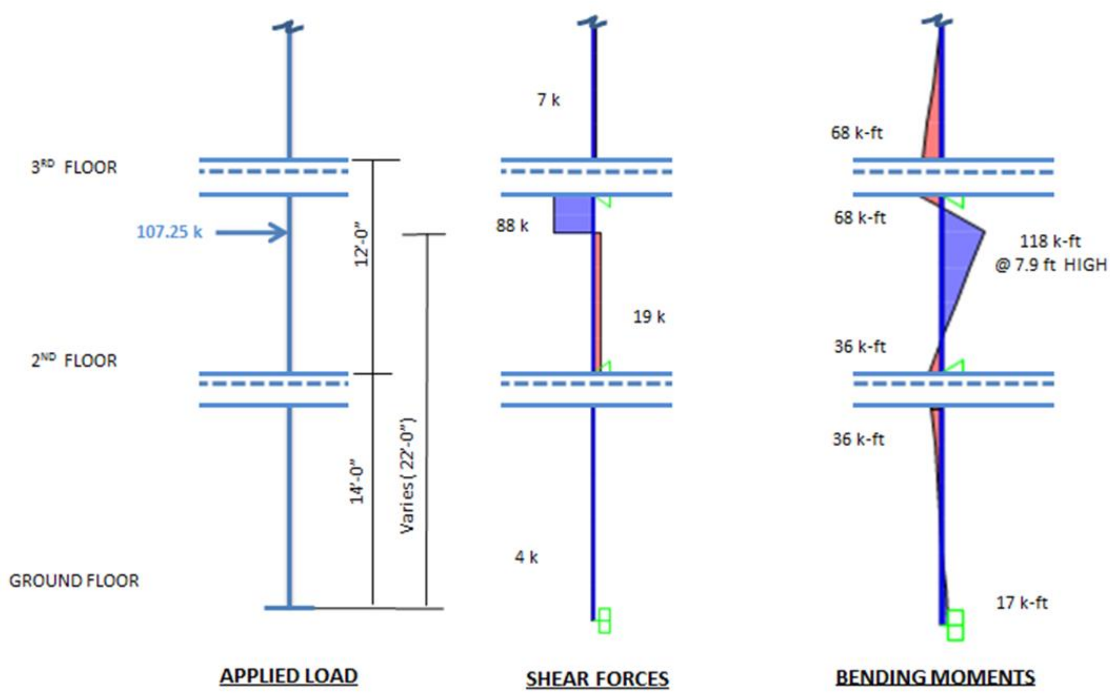

Figure C-35: Impact load applied at "d" away from the end of column on the 2nd floor

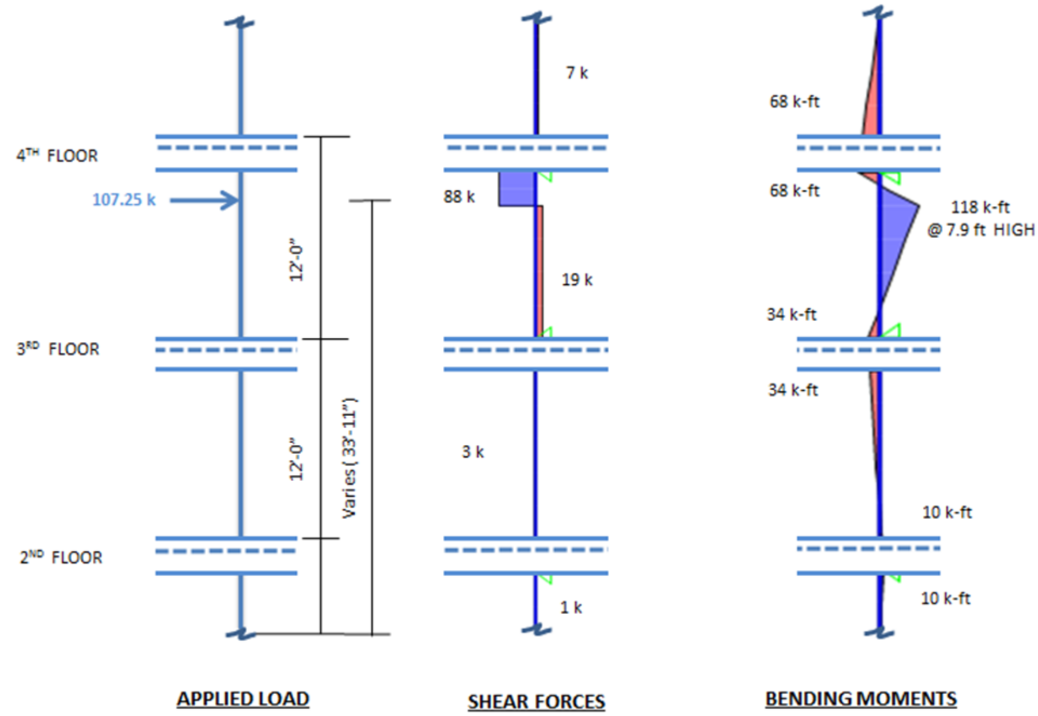

Figure C-36: Impact load applied at "d" away from the end of column on the 3rd floor

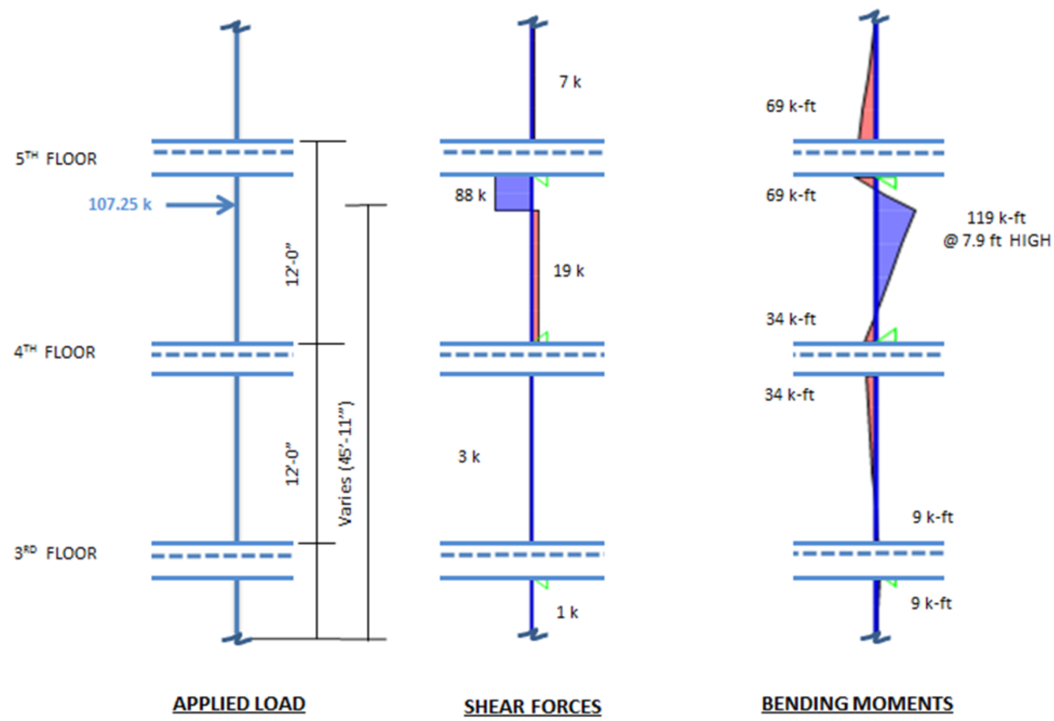

Figure C-37: Impact load applied at "d" away from the end of column on the 4th floor

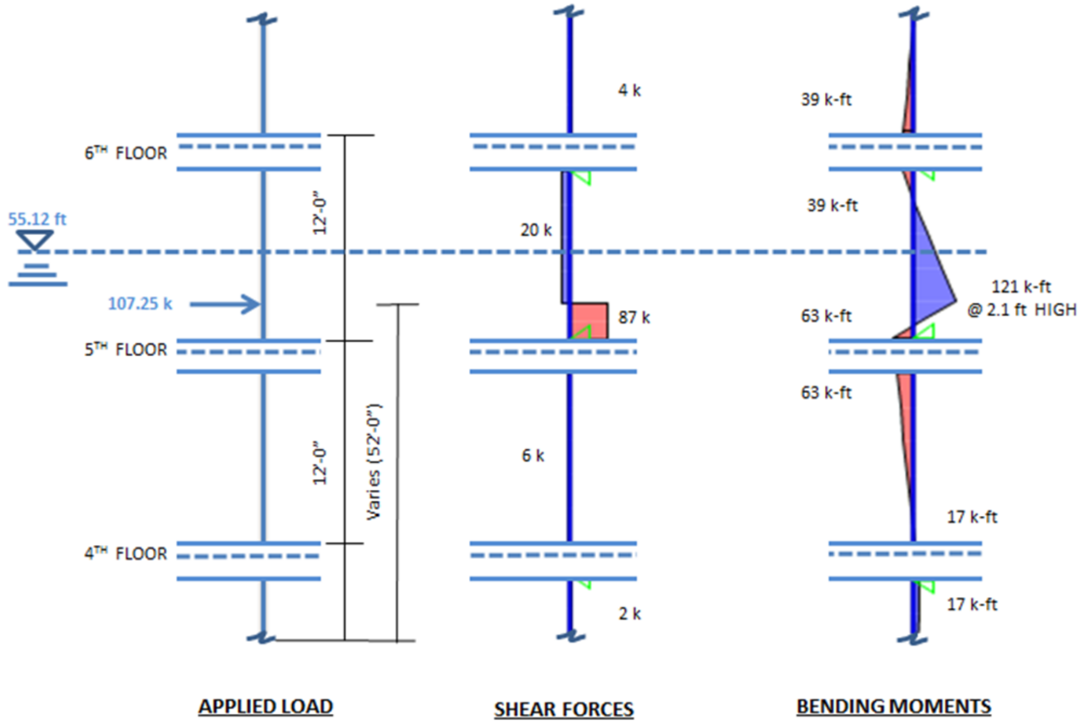

Figure C-38: Impact load applied at "d" away from the end of column on the 5th floor

Impact load at d + h_c:

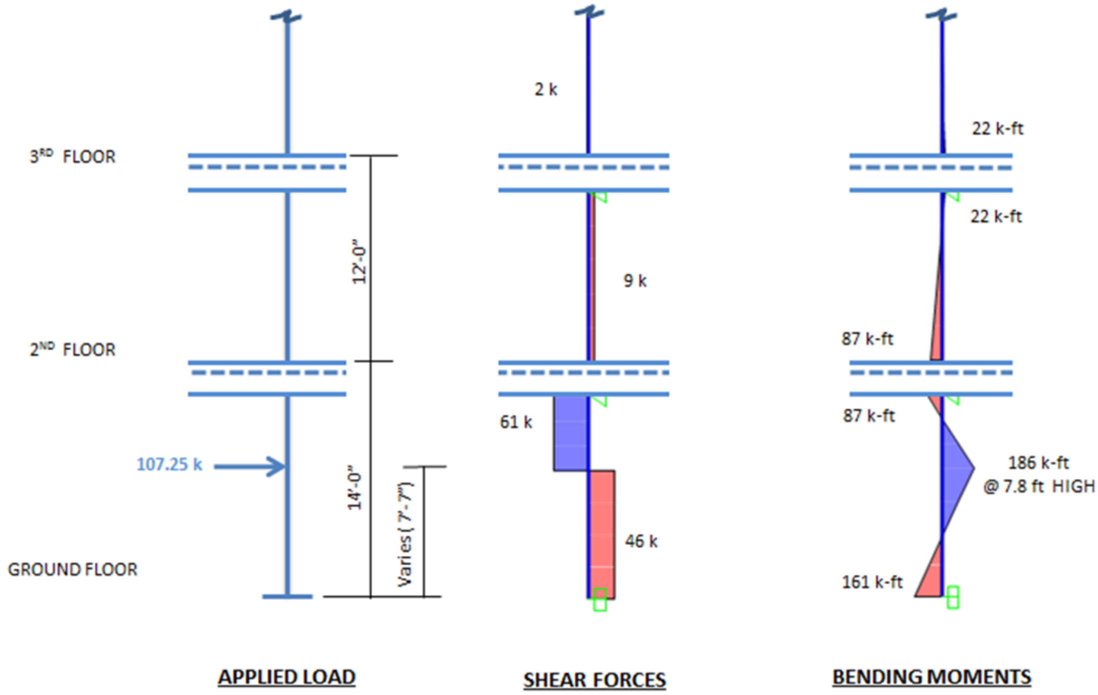

Figure C-39: Impact load applied at "d + h_c" away from the end of column on the ground floor

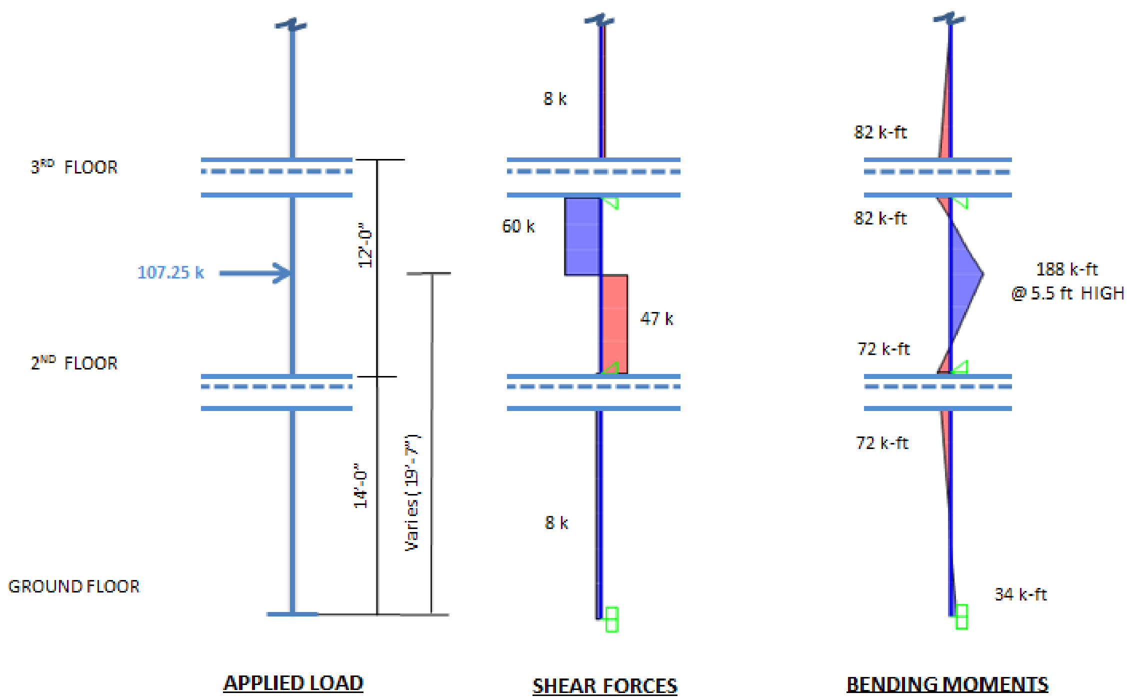

Figure C-40: Impact load applied at "d + h_c" away from the end of column on the 2nd floor

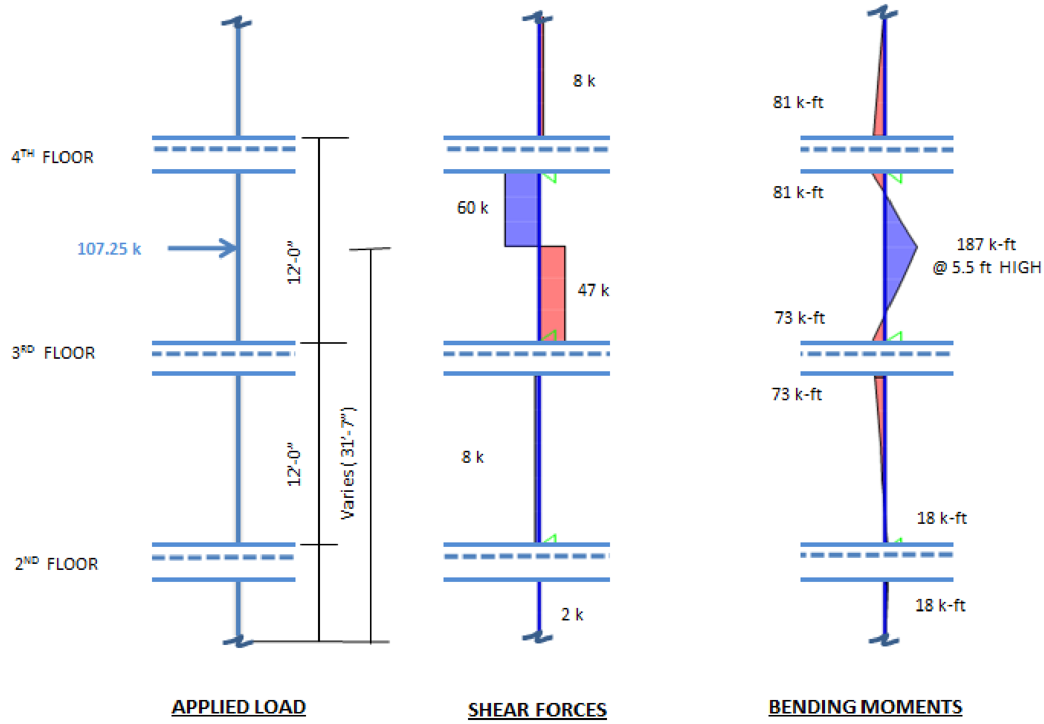

Figure C-41: Impact load applied at "d + h_c" away from the end of column on the 3rd floor

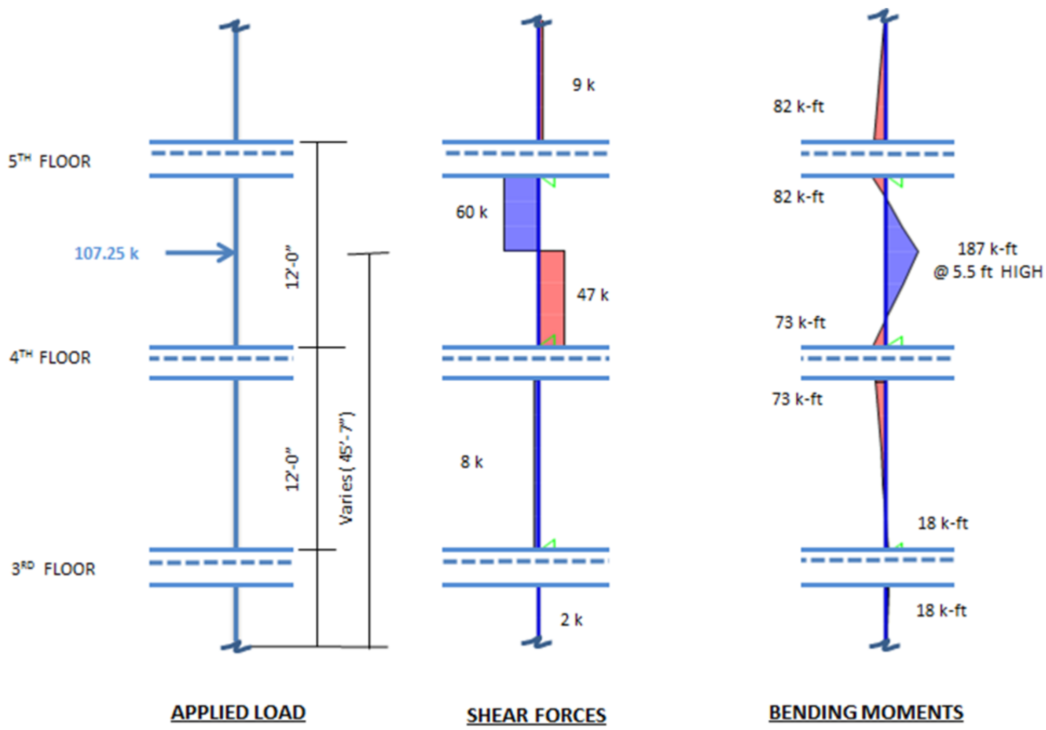

Figure C-42: Impact load applied at "d + h_c" away from the end of column on the 4th floor

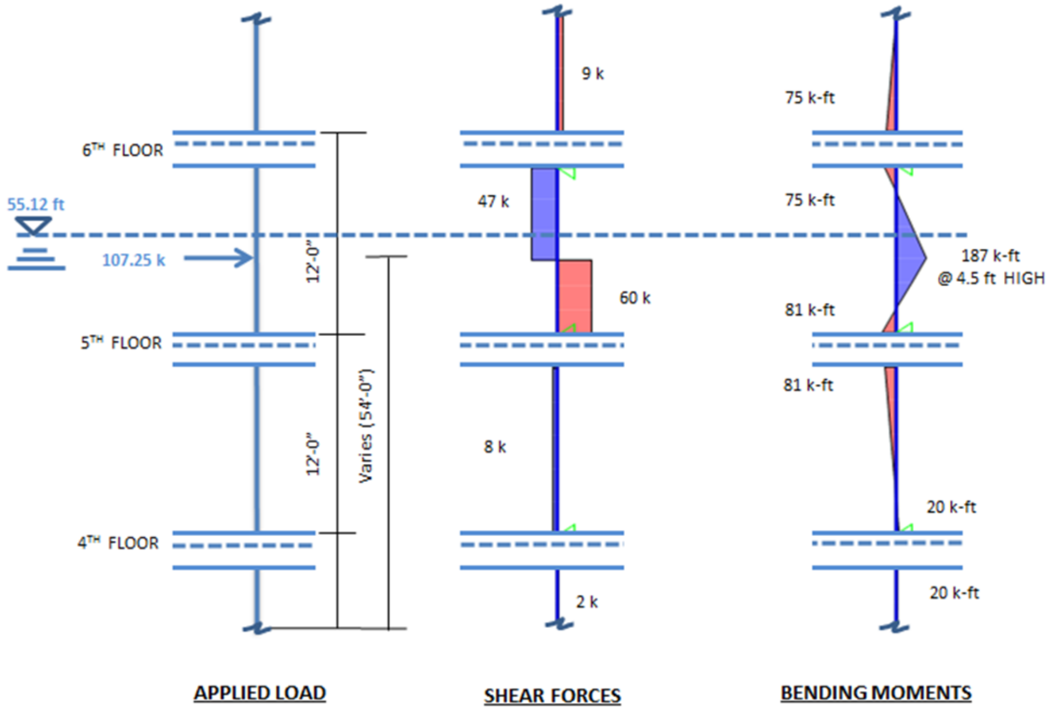

Figure C-43: Impact load applied at “d + h_c” away from the end of column on the 5th floor

Impact load at mid-height:

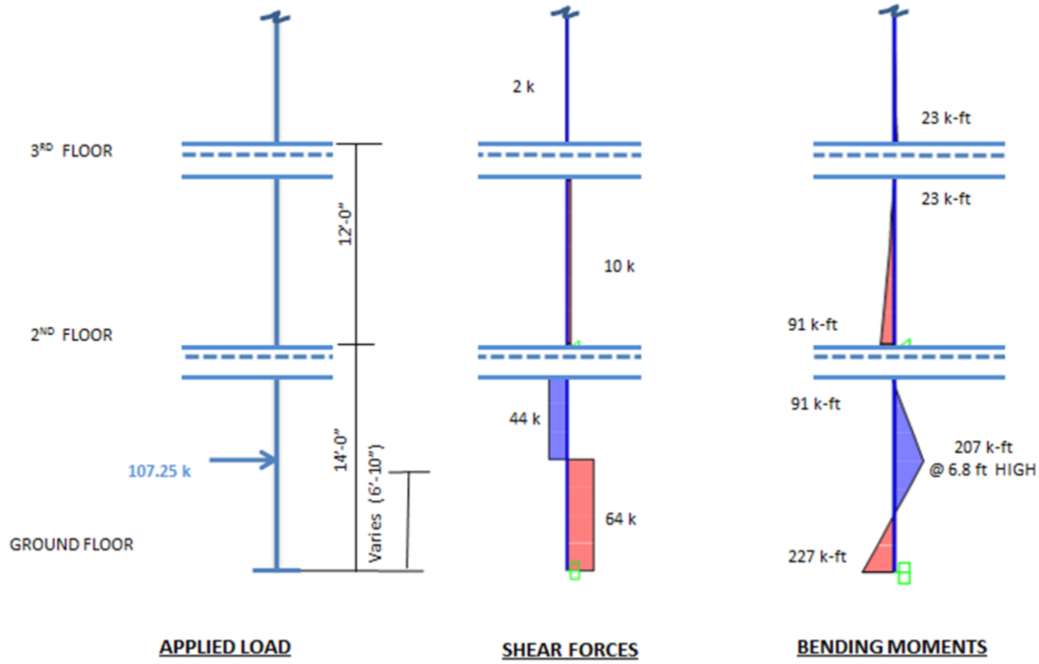

Figure C-44: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

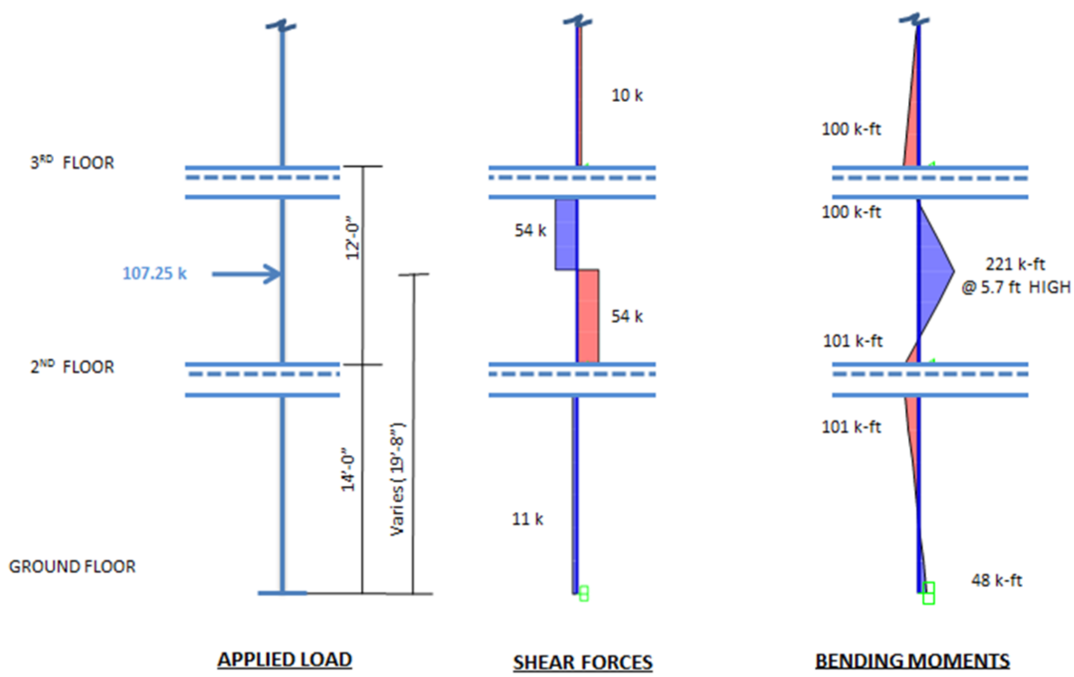

Figure C-45: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

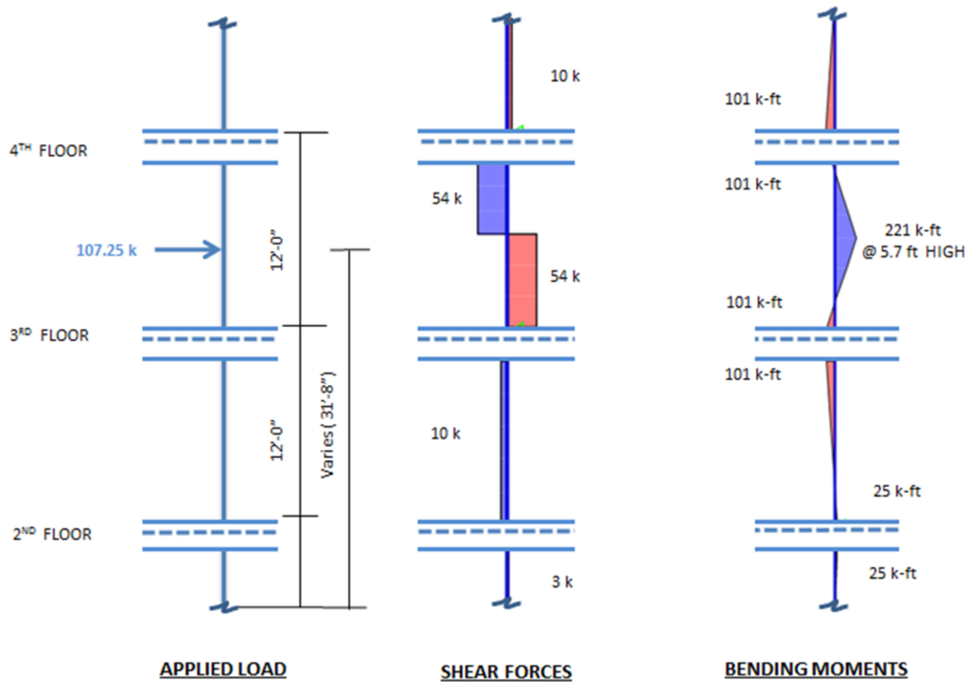

Figure C-46: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

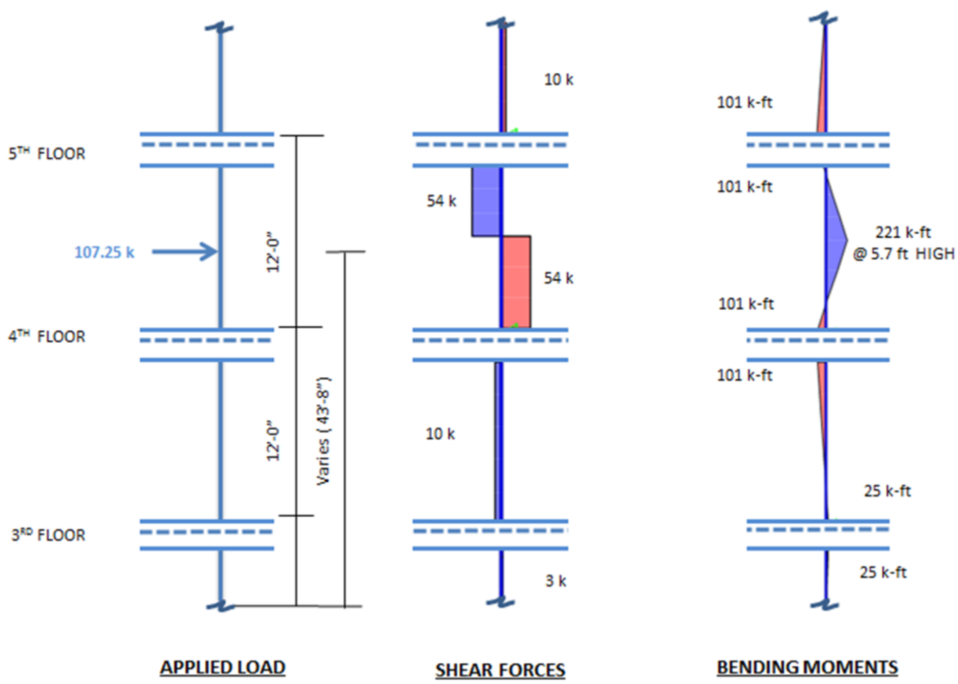

Figure C-47: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 4th floor column

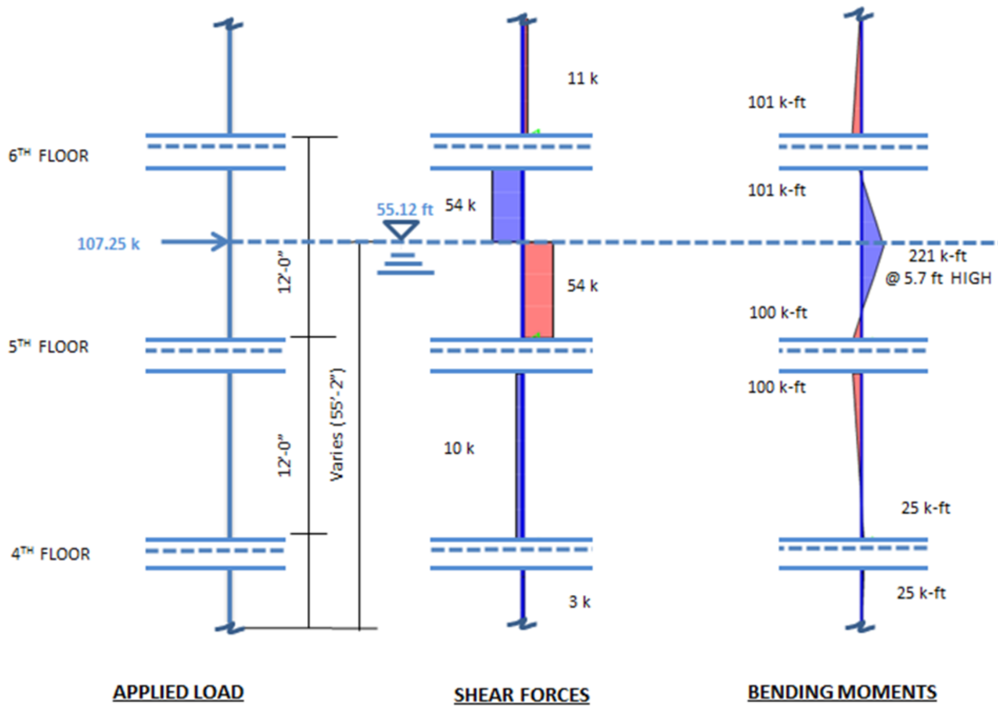

Figure C-48: Impact load applied at max water height due to max water height being lower than mid height mid-height of assumed lateral restraint points at top and bottom of the 5th floor column

Table C-4 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and log impact (Impact). In addition, because all of the exterior columns are part of the LFRS, **Table C-4** also lists the maximum axial load, bending moment and shear forces determined by the ETABS analysis for the modified base shear (Overall) (See **Section A.9.3**). These "Overall" systemic forces are then combined with the controlling component forces (either "Hydro" or "Impact") to obtain the "Combined" forces. Columns that are part of the transverse MRFs experience larger systemic loads and are therefore considered separately, along with columns having similar loads ("Special").

The original column designs will now be evaluated for these load combinations and modified if necessary.

Table C-4: Results from loading conditions of Hilo office building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
1572	502.3	469	248	1.2D+Ftsu+0.5L (Hydro)
1572	355.5	469	248	0.9D+Ftsu (Hydro)
227	502.3	88	61	1.2D+Ftsu+0.5L (Impact)
227	355.5	88	61	0.9D+Ftsu (Impact)
2856	-457.8	268	268	1.2D+Ftsu+0.5L (Overall)
2856	-604.5	268	268	0.9D+Ftsu (Overall)
4044	501.3	737	516	1.2D+Ftsu+0.5L (Combined)
4044	354.5	737	516	0.9D+Ftsu (Combined)
3642	-457.8	737	516	1.2D+Ftsu+0.5L (Combined, Corner)
3642	-604.5	737	516	0.9D+Ftsu (Combined, Corner)
Floor 2				
1271	418.5	340	119	1.2D+Ftsu+0.5L (Hydro)
1271	296.3	340	119	0.9D+Ftsu (Hydro)
221	418.5	88	60	1.2D+Ftsu+0.5L (Impact)
221	296.3	88	60	0.9D+Ftsu (Impact)
2326	-43.5	349	349	1.2D+Ftsu+0.5L (Overall)
2326	-165.7	349	349	0.9D+Ftsu (Overall)
2679	-43.5	689	468	1.2D+Ftsu+0.5L (Combined)
2679	-165.7	689	468	0.9D+Ftsu (Combined)
Floor 3				
1231	334.8	389	168	1.2D+Ftsu+0.5L (Hydro)
1231	237	389	168	0.9D+Ftsu (Hydro)
221	334.8	88	60	1.2D+Ftsu+0.5L (Impact)
221	237	88	60	0.9D+Ftsu (Impact)
2018	28.8	378	378	1.2D+Ftsu+0.5L (Overall)
2018	-69	378	378	0.9D+Ftsu (Overall)
2553	280.8	767	546	1.2D+Ftsu+0.5L (Combined)
2553	183	767	546	0.9D+Ftsu (Combined)
Floor 4				
561	251.1	58	58	1.2D+Ftsu+0.5L (Hydro)
561	177.8	58	58	0.9D+Ftsu (Hydro)
221	251.1	88	60	1.2D+Ftsu+0.5L (Impact)
221	177.8	88	60	0.9D+Ftsu (Impact)
255	243.1	46	46	1.2D+Ftsu+0.5L (Overall)
255	169.8	46	46	0.9D+Ftsu (Overall)
663	230.1	134	106	1.2D+Ftsu+0.5L (Combined)
663	156.8	134	106	0.9D+Ftsu (Combined)
Floor 5				
140	167.4	15	15	1.2D+Ftsu+0.5L (Hydro)
140	118.5	15	15	0.9D+Ftsu (Hydro)
221	167.4	87	60	1.2D+Ftsu+0.5L (Impact)
221	118.5	87	60	0.9D+Ftsu (Impact)
220	161.4	39	39	1.2D+Ftsu+0.5L (Overall)
220	112.5	39	39	0.9D+Ftsu (Overall)
441	161.4	126	99	1.2D+Ftsu+0.5L (Combined)
441	112.5	126	99	0.9D+Ftsu (Combined)
Floor 6				
35	83.7	4	4	1.2D+Ftsu+0.5L (Hydro)
35	59.3	4	4	0.9D+Ftsu (Hydro)
101	83.7	11	11	1.2D+Ftsu+0.5L (Impact)
101	59.3	11	11	0.9D+Ftsu (Impact)

C.11.1.2 Exterior Column Design for Combined Gravity and Tsunami Loads

The column chosen at Grid Intersection A-3 from **Figure C-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3** and listed in **Table C-4**. The column is considered braced against side-sway in the transverse direction.

Figure C-49 to **Figure C-55** show the interaction diagrams for a typical exterior column including the tsunami load combinations.

The blue solid line (Original Column Design Strength) represents the design strength for the original columns. The green dashed line (New Column Design Strength) represents the design strength needed if one were to take into account only the hydrodynamic and impact loads shown in **Figure C-32** to **Figure C-48**. The dotted red line (New Overall Column Design Strength) represents the design strength needed for taking into account only the overall building forces for each column shown in **Figure C-22** to **Figure C-26**. The orange dot-dashed line (New Combined Column Design Strength) represents the design strength needed for the overall loading combined with the hydrodynamic and impact loads per column. The light blue dotted-dashed line (New Orthogonal Column Design Strength) represents the columns at the intersection of two orthogonal moment frames designed for the combined loads. The forces applied to these columns are greater than those of the typical MRF columns. This series of plots is shown in alternating figures from **Figure C-49** to **Figure C-57** for all affected floor levels**.** Alternating **Figure C-50** to **Figure C-58** show the interaction diagrams for the combined forces with the controlling load combination for each column.

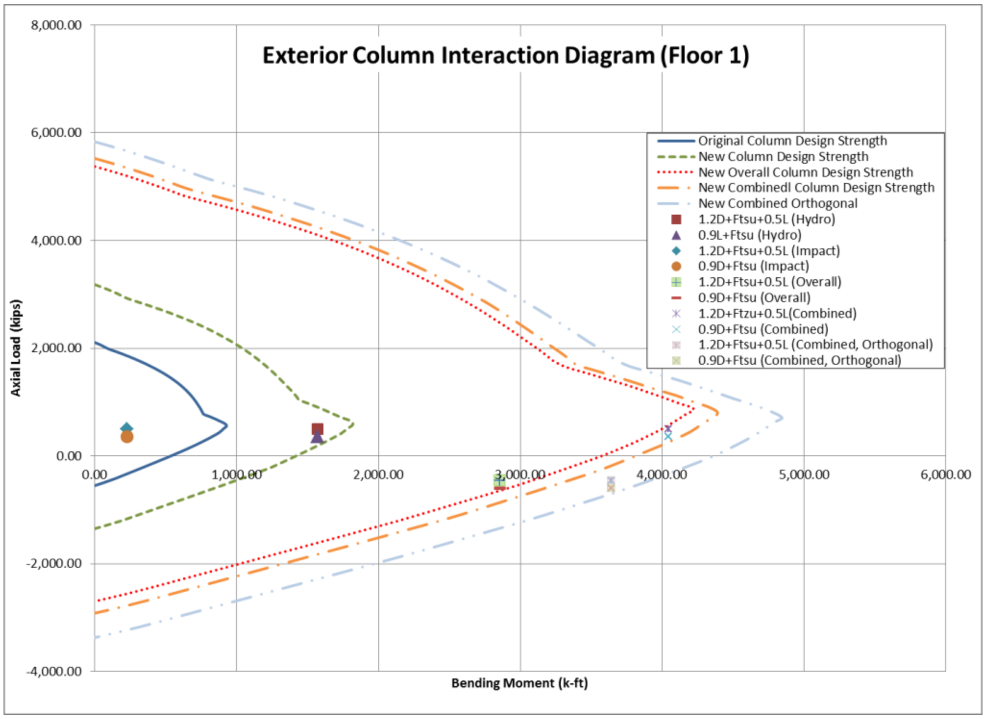

Figure C-49: Sequence of interaction diagrams for typical ground floor exterior column showing various tsunami load combinations

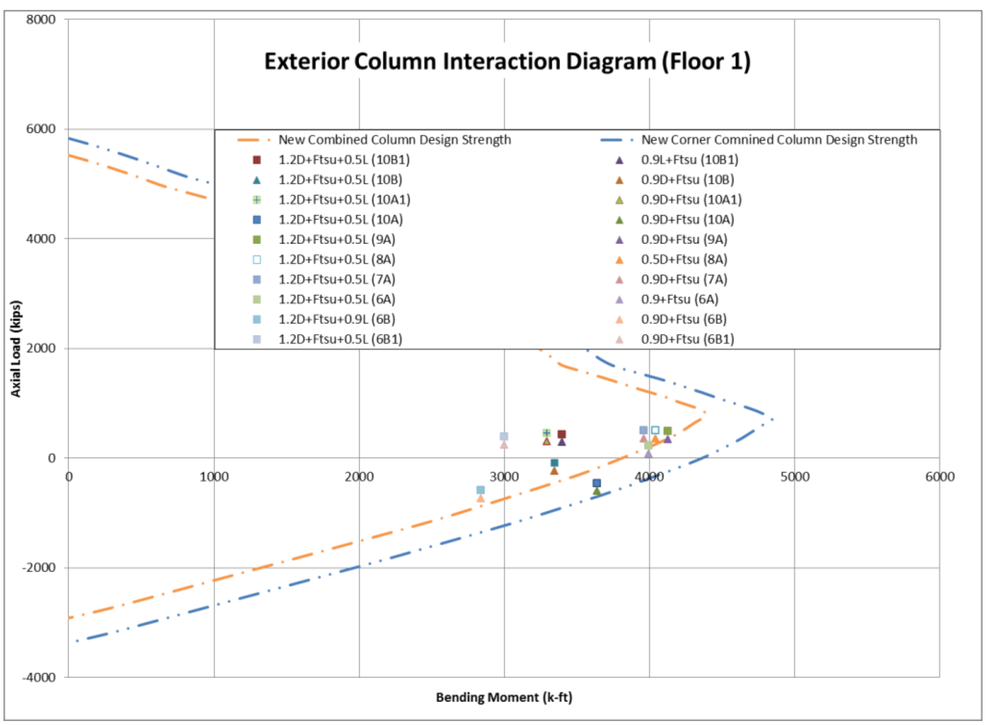

Figure C-50: Interaction diagrams for typical and special ground floor exterior column showing all combined tsunami load combinations

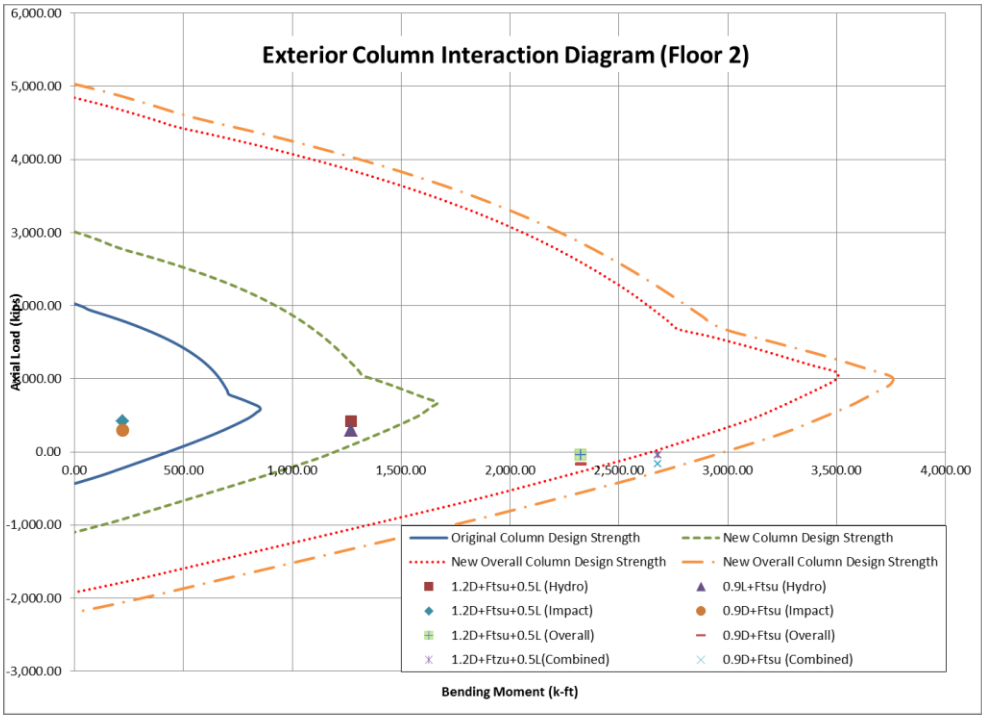

Figure C-51: Sequence of interaction diagrams for typical 2nd floor exterior column showing tsunami load combinations

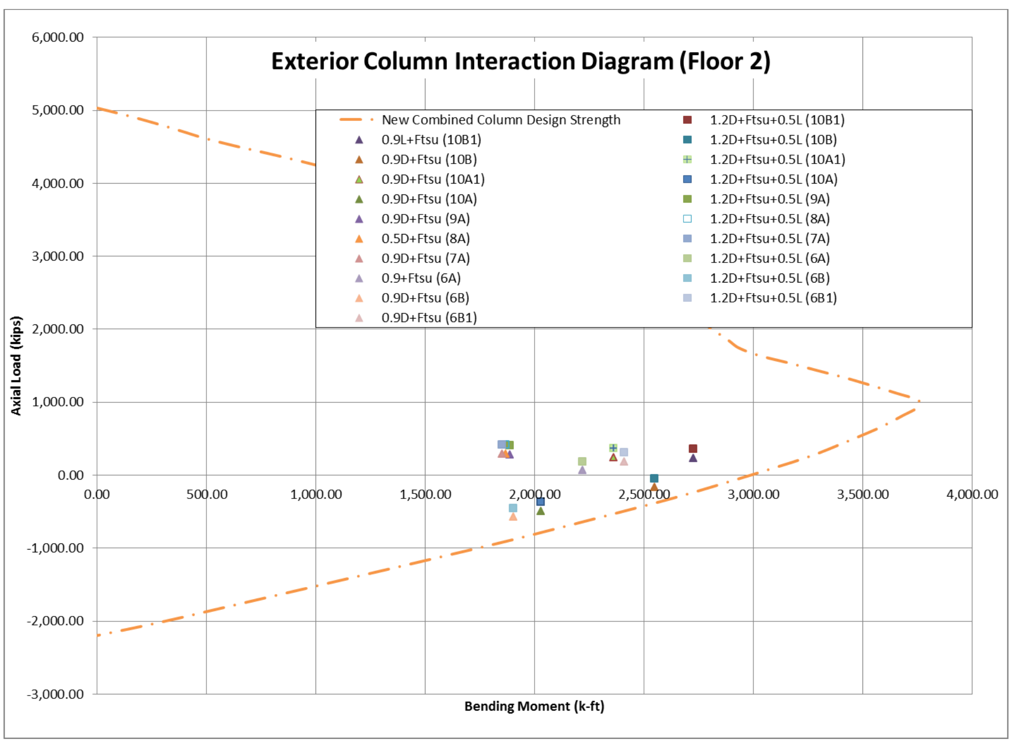

Figure C-52: Interaction diagrams for typical and special 2nd floor exterior column showing all combined tsunami load combinations

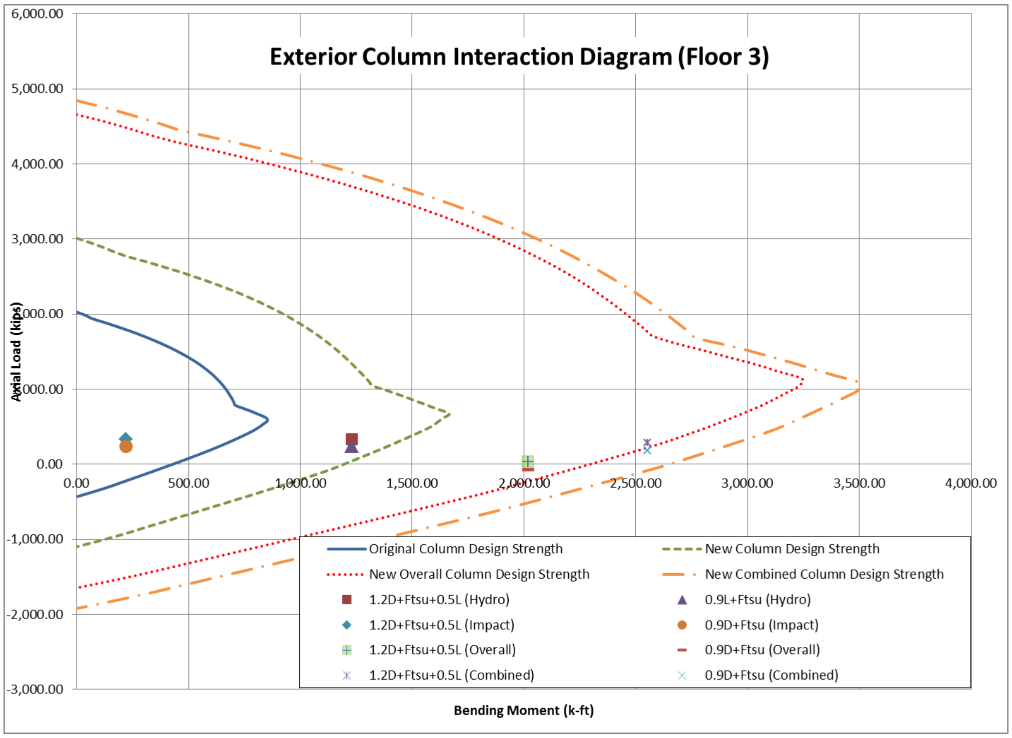

Figure C-53: Sequence of interaction diagrams for typical 3rd floor exterior column showing tsunami load combinations

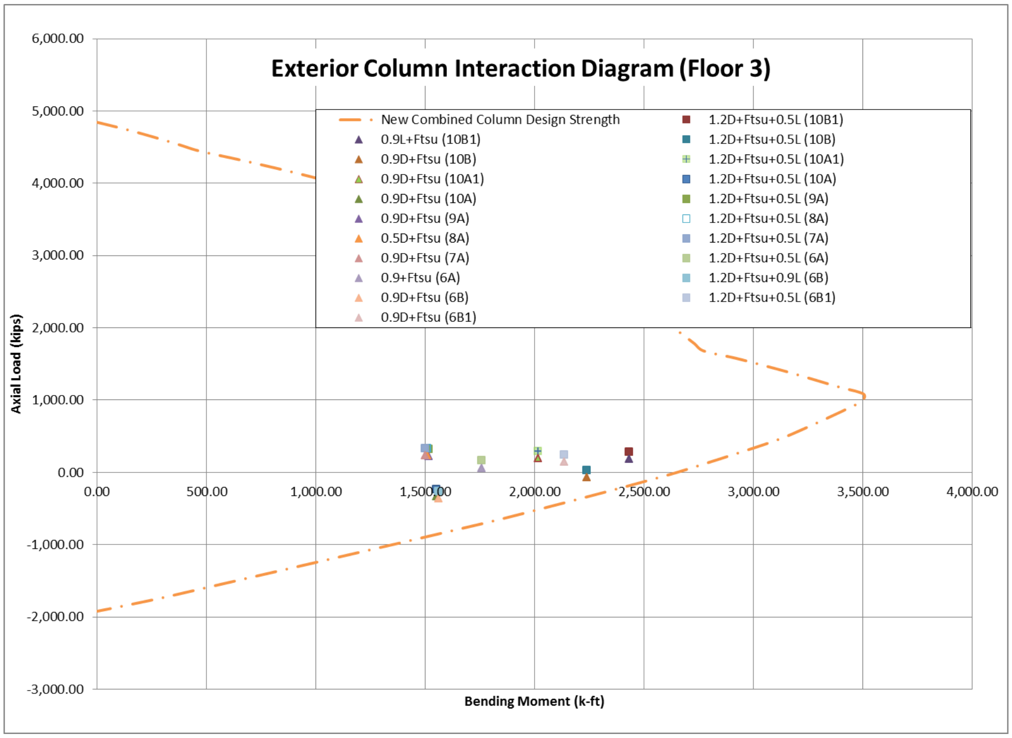

Figure C-54: Interaction diagrams for typical and special 3rd floor exterior column showing all combined tsunami load combinations

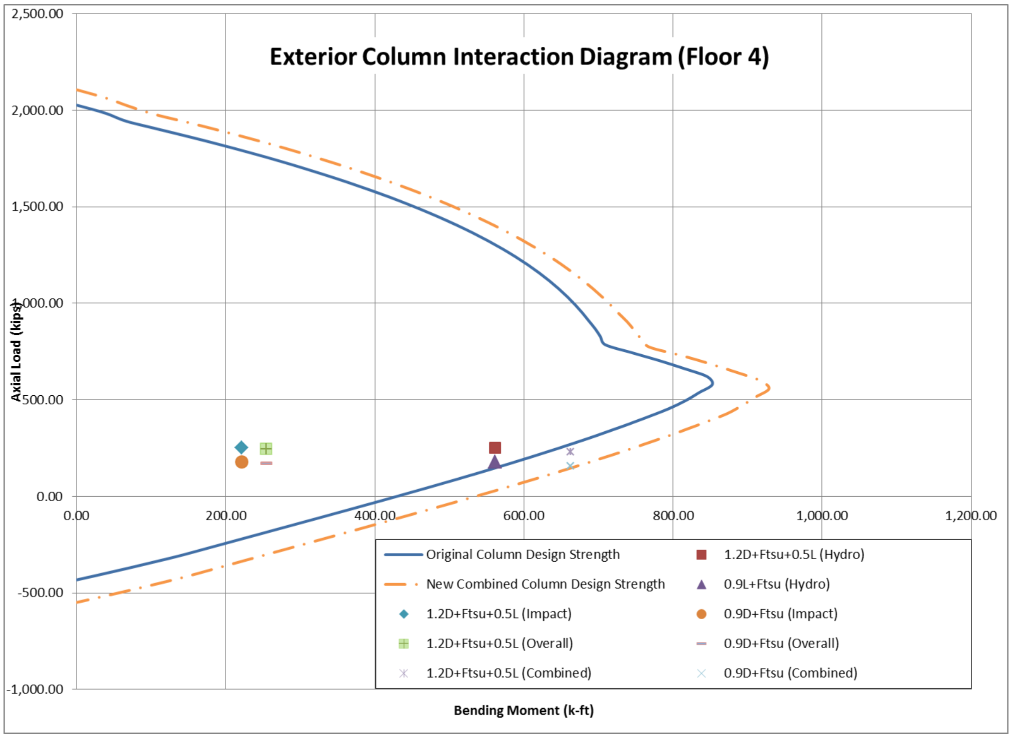

Figure C-55: Sequence of interaction diagrams for typical 4th floor exterior column showing various tsunami load combinations

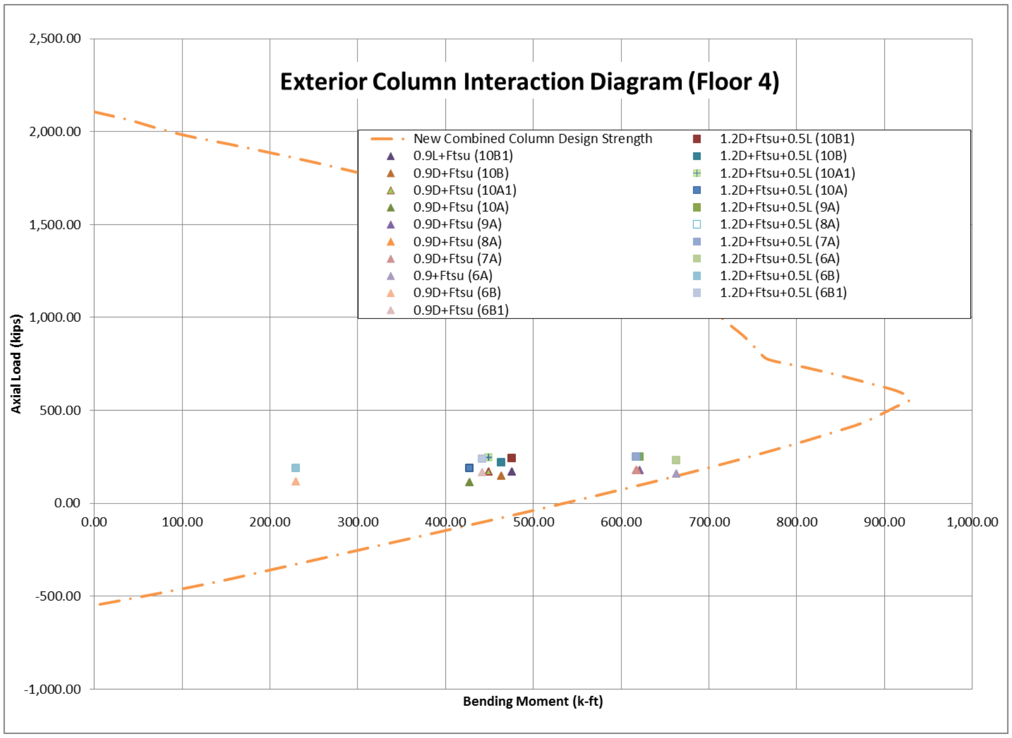

Figure C-56: Interaction diagrams for typical and special 4th floor exterior column showing all combined tsunami load combinations

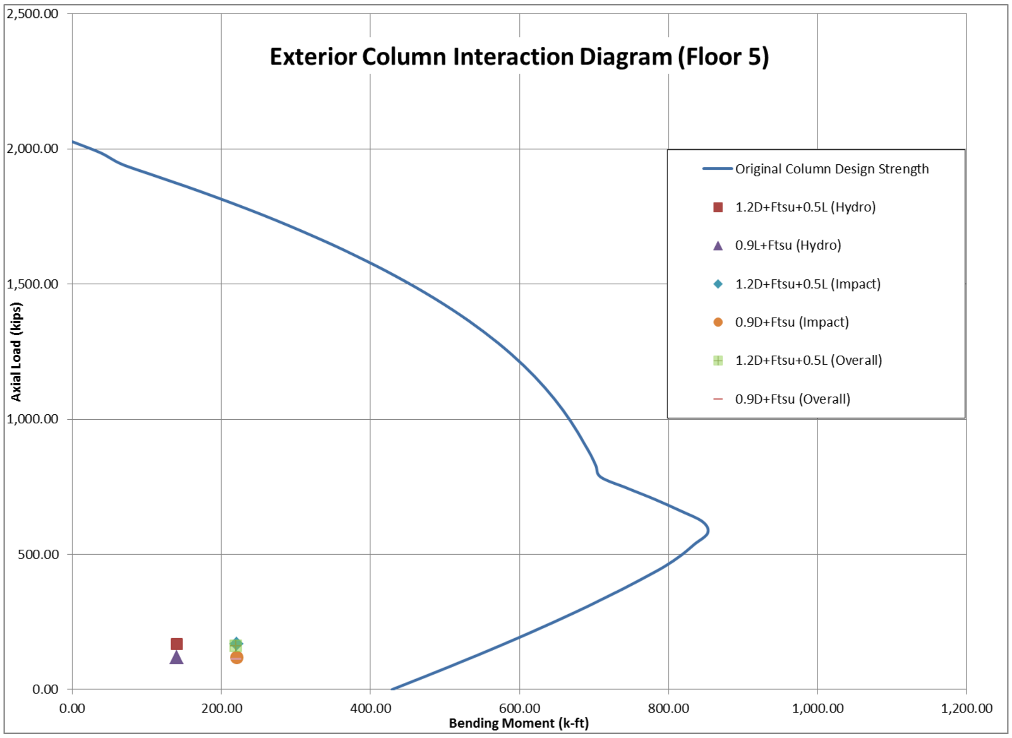

Figure C-57: Sequence of interaction diagrams for typical 5th floor exterior column showing various tsunami load combinations

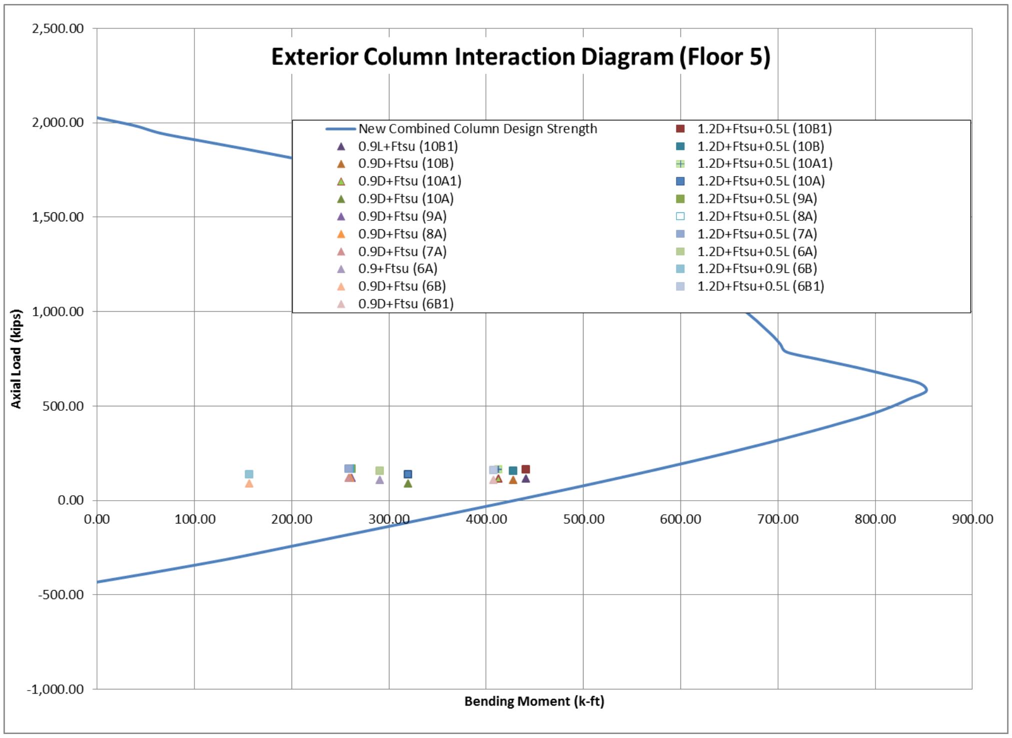

Figure C-58: Interaction diagrams for typical and special 5th floor exterior column showing all combined tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

C.11.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure C-49** to **Figure C-55** the original exterior columns are adequate for log impact loads, but the columns at the ground, 2nd, 3rd, and 4th floors must be strengthened to resist bending due to the combined hydrodynamic and overall system loads. Revised column designs shown in **Figure C-59** to **Figure C-66** were developed to satisfy the combined hydrodynamic and overall loads. The interaction diagrams for these new columns are shown in **Figure C-49** to **Figure C-53**. The ties in these columns are designed in Section A.11.1.4 for the applied tsunami shear forces.

Floor 1

End Section (A)

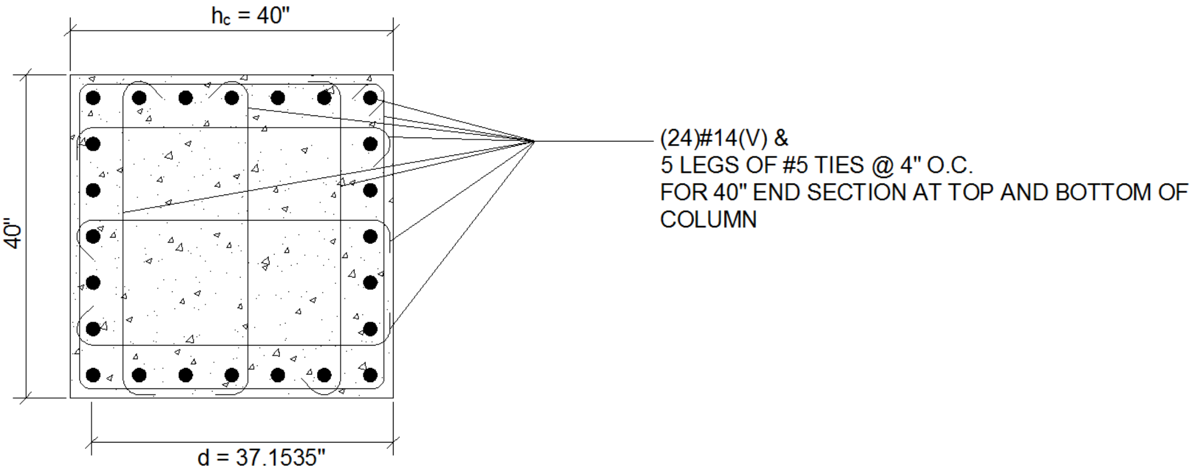

Figure C-59: Exterior column, cross-section at end section of column at ground floor level based on tsunami design requirements.

Center Section (B)

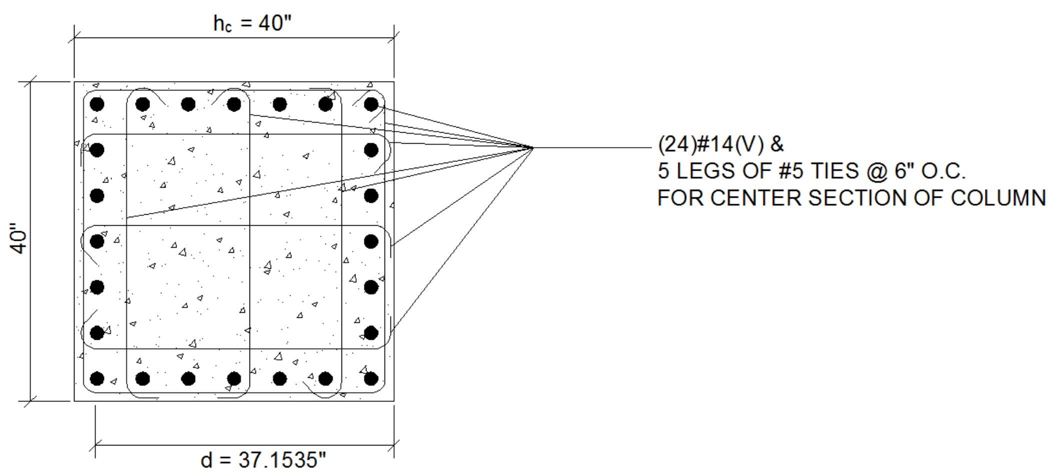

Figure C-60: Exterior column, cross-section at center section of column at ground floor level based on tsunami design requirements.

Floor 1 Corner

End Section (A)

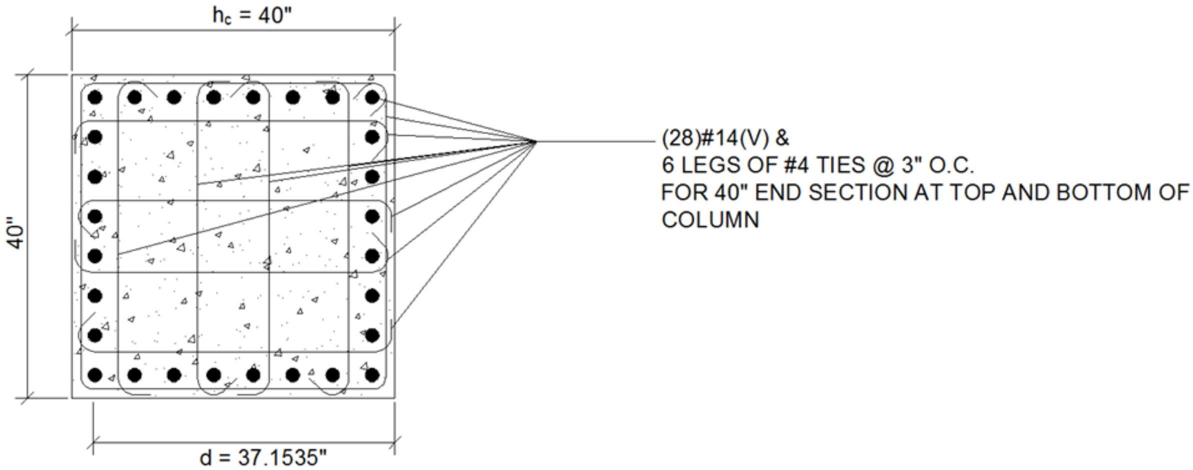

Figure C-61: Exterior corner column, cross-section at end section of column at ground floor level based on tsunami design requirements.

Center Section (B)

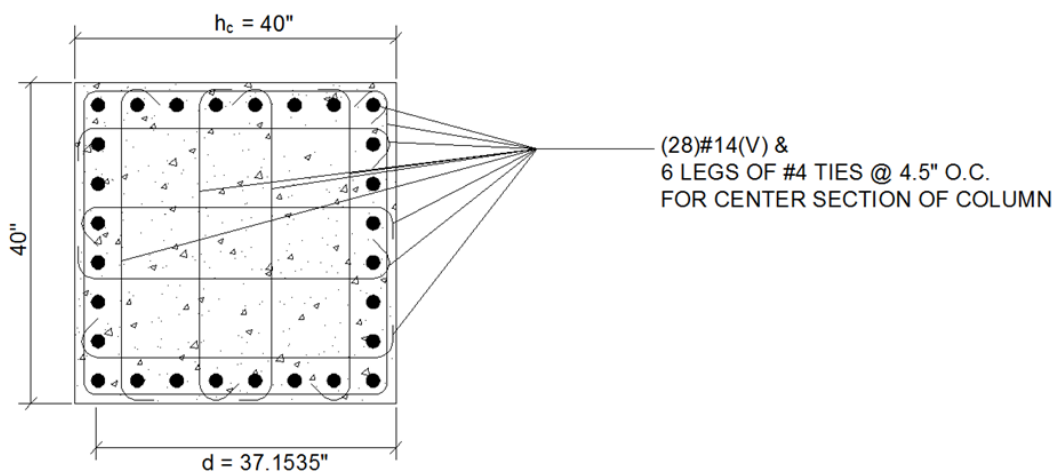

Figure C-62: Exterior corner column, cross-section at center section of column at ground floor level at the corners based on tsunami design requirements.

Floor 2

End Section (A)

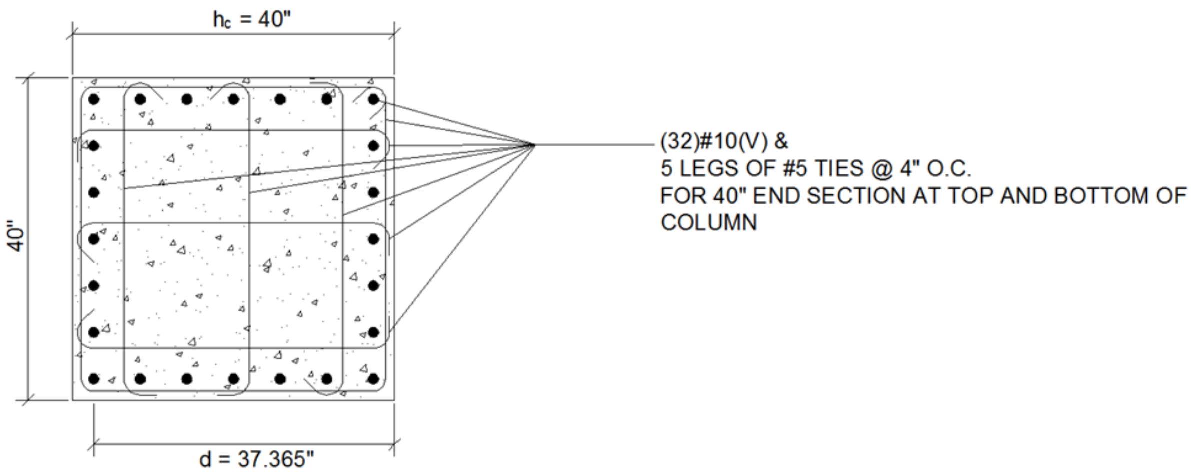

Figure C-63: Exterior column, cross section at end section of column at the 2nd floor level based on tsunami design requirements.

Center Section (B)

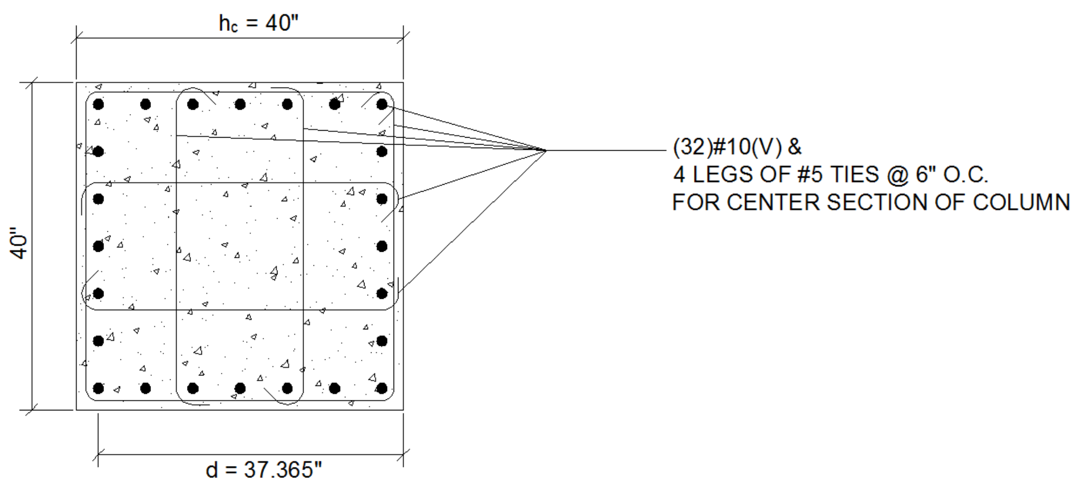

Figure C-64: Exterior column, cross-section at center section of column at the 2nd floor level based on tsunami design requirements.

Floor 3

End Section (A)

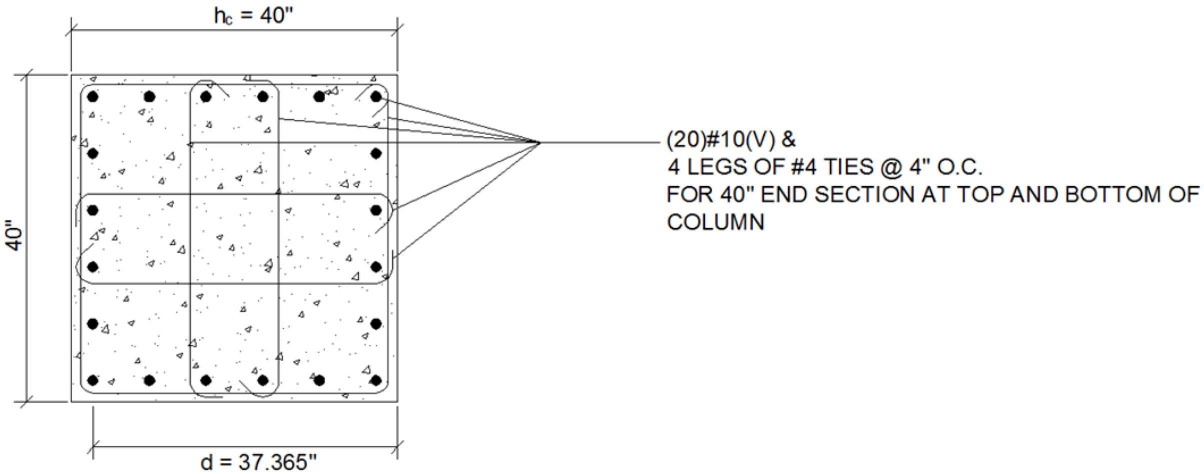

Figure C-65: Exterior column, cross-section at end section of column at the 3rd floor level based on tsunami design requirements.

Center Section (B)

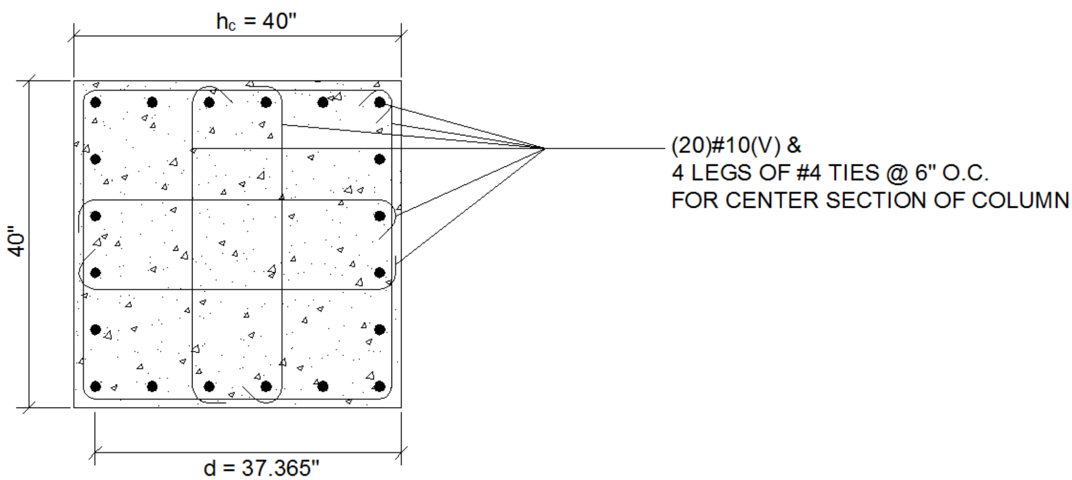

Figure C-66: Exterior column, cross-section at center section of column at the 3rd floor level based on tsunami design requirements.

Floor 4

End Section (A)

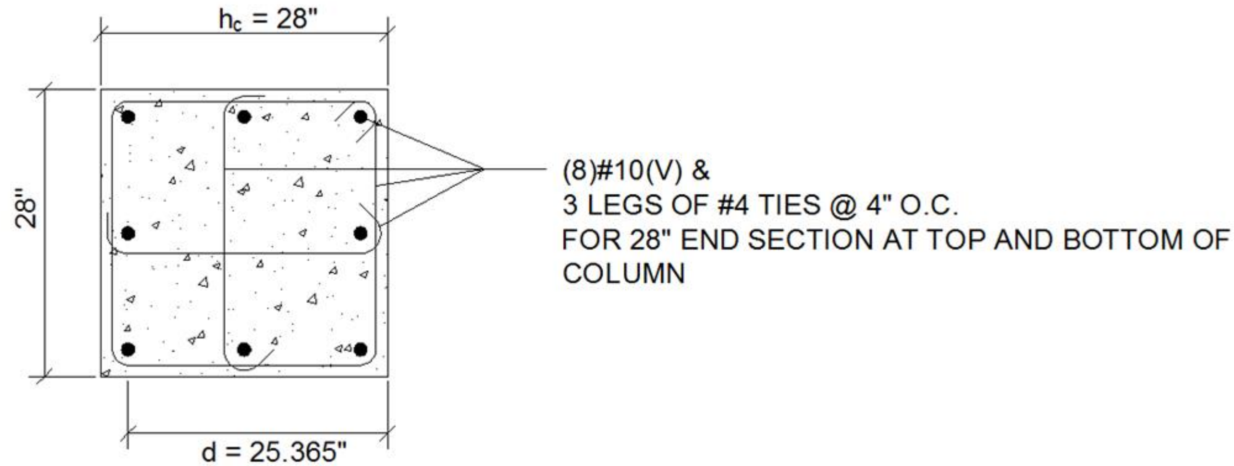

Figure C-67: Exterior column, cross-section at end section of column at the 4th floor level based on tsunami design requirements.

Center Section (B)

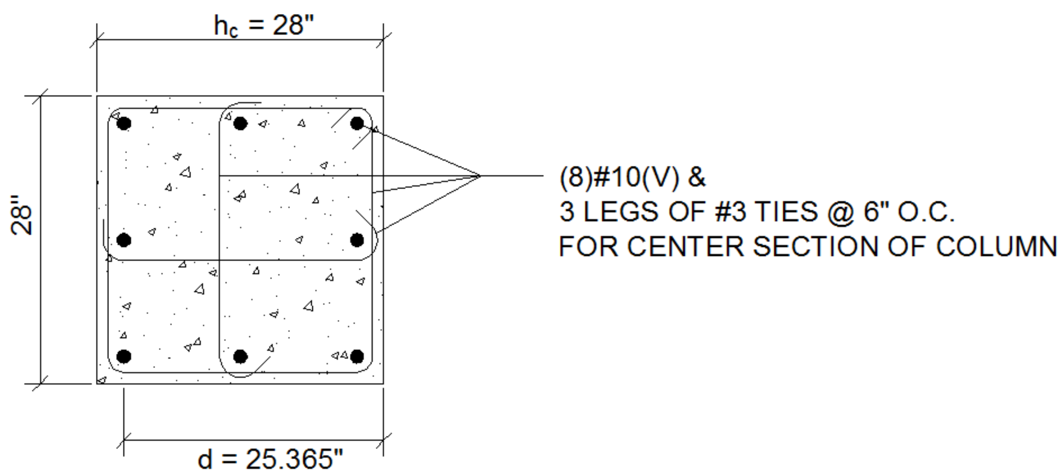

Figure C-68: Exterior column, cross-section at center section of column at the 4th floor level based on tsunami design requirements.

C.11.1.4 Exterior Column Shear Design

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = 354 kips.

The shear capacities of the 40”x40” columns with 5 leg #5 Stirrups at 4” o.c. in the end section and 5 leg #5 Stirrups at 6” o.c. in the center section are given by:

$\varnothing V_n = \varnothing(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{-354{,}500}{2{,}000\times 40\times 40}\right)40\times 37.1535/1{,}000$ = 221 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(5\times 0.31)\times 60{,}000\times 37.1535}{4\times 1{,}000}$ = 864 kips

$$V_S = \frac{A_v f_y d}{s} = 864 \text{ kips} \ngtr 8\sqrt{f_C{}'}\ \mathrm{b\ d} = 8\times\sqrt{4{,}500}\times 40\times 37.1535 = 798 \text{ kips} \therefore \text{use } 798 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(5\times 0.31)\times 60{,}000\times 37.1535}{6\times 1{,}000}$ = 576 kips.

$$V_S = \frac{A_v f_y d}{s} = 576 \text{ kips} \ngtr 8\sqrt{f_C{}'}\ \mathrm{b\ d} = 8\times\sqrt{4{,}500}\times 40\times 37.1535 = 798 \text{ kips} \therefore \text{use } 576 \text{ kips}$$

Therefore in the end sections, $\varnothing V_n$ = 0.75 (221 + 798) = 764 k

and in the center sections, $\varnothing V_n$ = 0.75 (221 + 575) = 598 k.

At d: V_u = 737 k < ϕV_n = 764 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$*:* V_u = 516 k < ϕV_n = 598 k, therefore the column is adequate for shear at the center section.

Critical Shears in Corner Columns at 1st Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = -604.5 kips.

The shear capacities of the 40”x40” column with 6 leg #4 Stirrups at 3” o.c. in the end sections and 6 leg #4 Stirrups at 4.5” o.c. in the center section are given by:

$\varnothing V_n = \varnothing(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{5{,}000}\left(1+\frac{-604{,}500}{2{,}000\times 40\times 40}\right)40\times 37.1535/1{,}000$ = 152 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(6\times 0.2)\times 60{,}000\times 37.1535}{3\times 1{,}000}$ = 892 kips

$$V_S = \frac{A_v f_y d}{s} = 892 \text{ kips} \ngtr 8\sqrt{f_C{}'}\ \mathrm{b\ d} = 8\times\sqrt{5{,}000}\times 40\times 37.1535 = 841 \text{ kips} \therefore \text{use } 841 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(6\times 0.2)\times 60{,}000\times 37.1535}{4.5\times 1{,}000}$ = 594 kips

$$V_S = \frac{A_v f_y d}{s} = 594 \text{ kips} \ngtr 8\sqrt{f_C{}'}\ \mathrm{b\ d} = 8\times\sqrt{5{,}000}\times 40\times 37.1535 = 841 \text{ kips} \therefore \text{use } 594 \text{ kips}$$

Therefore in the end sections, $\varnothing V_n$ = 0.75 (170 + 841) = 758 k

and in the center sections, $\varnothing V_n$ = 0.75 (170 + 594) = 574 k

At d: V_u = 737 k < ϕV_n = 758 k, therefore the column is adequate for shear at the end sections.

At d + h_c*:* V_u = 516 k < ϕV_n = 574 k, therefore the column is adequate for shear at the center section

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b,** P_u = -165.5 k:

The shear capacities of the 40"x40" column with 5 leg #5 stirrups at 4" o.c. in the end sections and 4 leg #5 stirrups at 6" o.c. in the center section are given by:

$$ØV_n = Ø(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4{,}500}\left(1+\frac{-165{,}500}{2{,}000\times40\times40}\right)40\times37.365/1{,}000$ = 190 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(5\times0.31)\times60{,}000\times37.365}{4\times1{,}000}$ = 869 kips

$$V_S = \frac{A_v f_y d}{s} = 892 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}500}\times40\times37.365 = 802 \text{ kips} \therefore \text{use } 802 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(5\times0.2)\times60{,}000\times37.365}{6\times1{,}000}$ = 463 kips.

$$V_S = \frac{A_v f_y d}{s} = 463 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}500}\times40\times37.365 = 802 \text{ kips} \therefore \text{use } 463 \text{ kips}$$

Therefore in the end section, $ØV_n$ = 0.75 (190 + 802) = 744 k

and in the center section, $ØV_n$ = 0.75 (190 + 463) = 490 k

At d: V_u = 689 k < ϕV_n = 744 k, therefore the column is adequate for shear at the edge.

At d + h_c*:* V_u = 468 k < ϕV_n = 490 k, therefore the column is adequate for shear at the center

Critical Shears in Columns at 3rd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b,** P_u = 183 k:

The shear capacities of the 40"x40" column with 5 leg #5 stirrups at 4" o.c. in the end sections and 5 leg #5 stirrups at 6" o.c. in the center section are given by:

$$ØV_n = Ø(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4{,}500}\left(1+\frac{183{,}000}{2{,}000\times40\times40}\right)40\times37.365/1{,}000$ = 212 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(5\times0.31)\times60{,}000\times37.365}{4\times1{,}000}$ = 869 kips

$$V_S = \frac{A_v f_y d}{s} = 869 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\text{b d} = 8\times\sqrt{4{,}500}\times40\times37.365 = 802 \text{ kips} \therefore \text{use } 802 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(5\times0.31)\times60{,}000\times37.365}{6\times1{,}000}$ = 579 kips.

$$V_S = \frac{A_v f_y d}{s} = 892 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\text{b d} = 8\times\sqrt{4{,}500}\times40\times37.365 = 802 \text{ kips} \therefore \text{use } 579 \text{ kips}$$

Therefore in the end section, $\varnothing V_n$ = 0.75 (200 + 869) = 802 k

and in the center section, $\varnothing V_n$ = 0.75 (211 + 579) = 593 k

At d: V_u = 767 k ≤ ϕV_n = 761 k, ϕV_n and V_u < 1% different and therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 546 k < ϕV_n = 593 k, therefore the column is adequate for shear at the center

Critical Shears in Columns at 4th Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b,** P_u = 156.8 k:

The shear capacities of the 28"x28" column with 3 leg #4 stirrups at 4" o.c. in the end sections and 3 leg #3 stirrups at 6" o.c. in the center section are given by:

$$\varnothing V_n = \varnothing(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{156{,}800}{2{,}000\times28\times28}\right)28\times25.365/1{,}000$ = 99 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times25.365}{4\times1{,}000}$ = 228 kips

$$V_S = \frac{A_v f_y d}{s} = 228 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\text{b d} = 8\times\sqrt{4{,}000}\times40\times37.365 = 359 \text{ kips} \therefore \text{use } 228 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times25.365}{6\times1{,}000}$ = 84 kips.

$$V_S = \frac{A_v f_y d}{s} = 84 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\text{b d} = 8\times\sqrt{4{,}000}\times40\times37.365 = 359 \text{ kips} \therefore \text{use } 84 \text{ kips}$$

Therefore in the end section, $\varnothing V_n$ = 0.75 (99 + 228) = 245 k

and in the center section, $\varnothing V_n$ = 0.75 (99 + 84) = 137 k

At d: V_u = 134 k < ϕV_n = 245 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 106 k < ϕV_n = 137 k, therefore the column is adequate for shear at the center

Critical Shears in Columns at 5th Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b,** P_u = 112.5 k:

The shear capacities of the 28”x28” column with 3 leg #4 stirrups at 4” o.c. in the end sections and 3 leg #3 stirrups at 6” o.c. in the center section are given by:

$$\emptyset V_n = \emptyset(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{112{,}500}{2{,}000\times28\times28}\right)28\times25.465/1{,}000$ = 97 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times25.465}{4\times1{,}000}$ = 229 kips

$$V_S = \frac{A_v f_y d}{s} = 229 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}000}\times40\times37.465 = 360 \text{ kips} \therefore \text{use } 229 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times25.465}{6\times1{,}000}$ = 84 kips.

$$V_S = \frac{A_v f_y d}{s} = 84 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}000}\times40\times37.465 = 360 \text{ kips} \therefore \text{use } 84 \text{ kips}$$

Therefore in the end section, $\emptyset V_n$ = 0.75 (97 + 229) = 244 k

and in the center section, $\emptyset V_n$ = 0.75 (97 + 84) = 136 k

At d: V_u = 126 k < ϕV_n = 244 k, therefore the column is adequate for shear at the edge.

At $d + h_c$: V_u = 99 k < ϕV_n = 136 k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear force.

C.11.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure C-16.** The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 24 in square column cross section shown in **Figure C-69** and **Figure C-70** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force for the end section of the column occurs at a distance “*d*” from the ends of the column, where d = 24 - 1.5 - 0.5 - 0.5 = 21.5 in. The critical shear force for the center section of the column occurs at “$d + h_c$” from the end of the column, where $d + h_c$ = 21.5 + 24 = 45.5 in.

Floor 1 – 6

End Section (A)

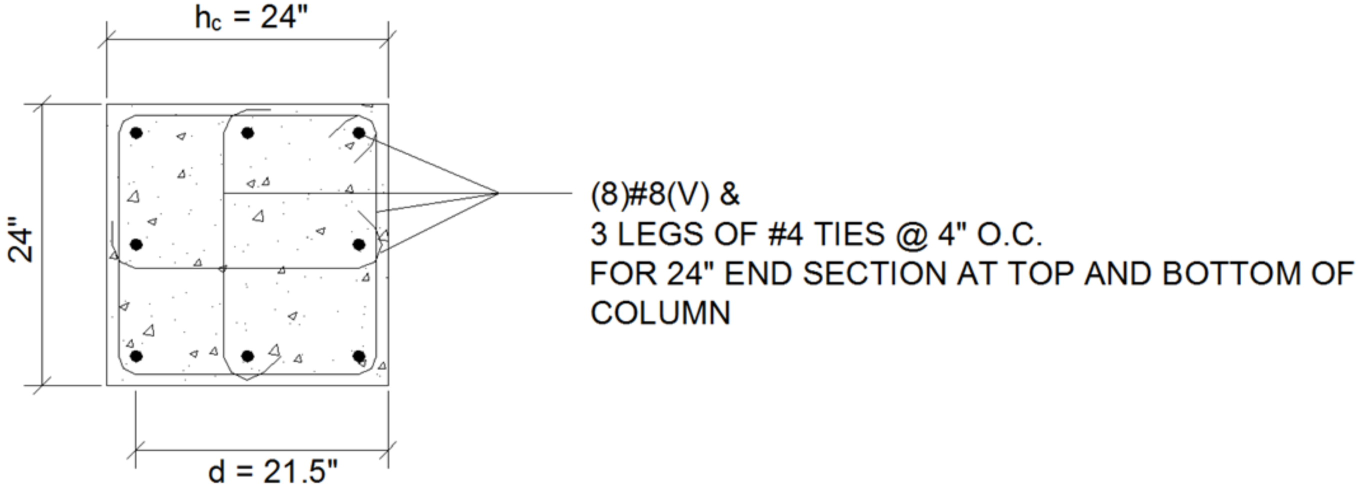

Figure C-69: Interior column, end section cross-section for column at all floor levels based on SDC D design.

Center Section (B)

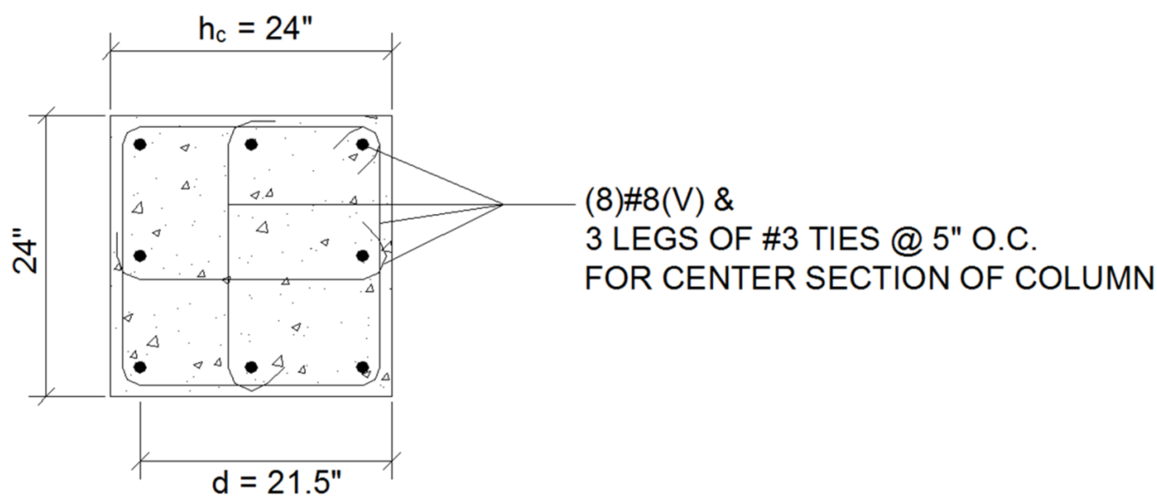

Figure C-70: Interior column, center section cross-section for column at all floor levels based on SDC D design.

C.11.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 29 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$$P_D = [120(28)(29)(6) + 2^2(150)(74)]/1000 = 629 \text{ k.}$$

Floor Live load reduction factor = $0.25+15/[4(29)(28)(5)]^{0.5} = 0.367$, therefore using 0.4 gives:

$P_L = 0.4[95(5)+65(24)](28)(5)]/1000 = 114$ *k.*

Roof Live Load reduction factor = $R_1R_2= 0.6(1.0) = 0.6$ for $A_t > 600$ sf, therefore the roof live load is:

$P_{Lr} = 0.6(20)(28)(29) = 9.7$ *k*

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

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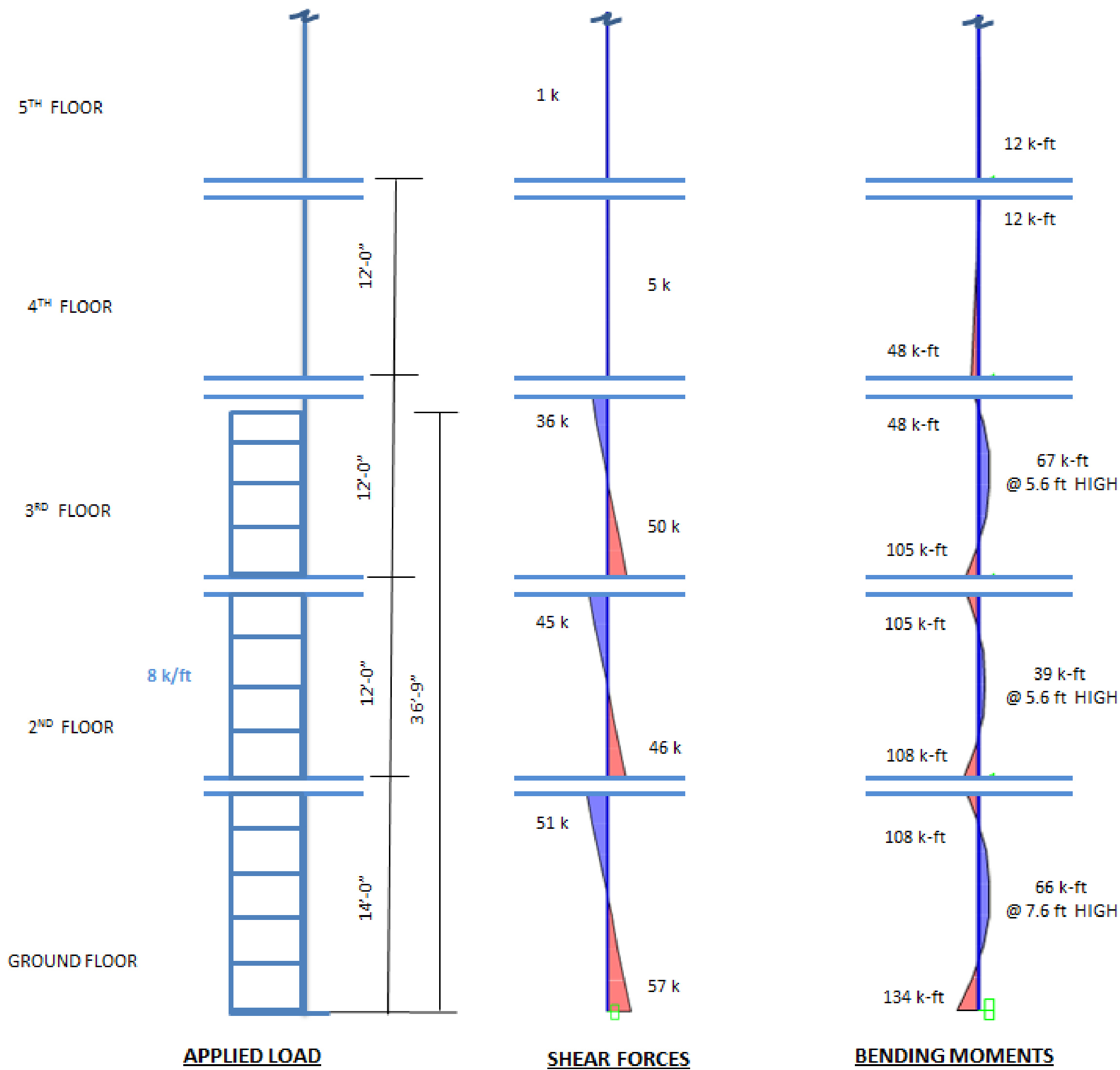


Figure C-71: Hydrodynamic loading on interior column of Hilo office building due to Load Case 2

Table C-5 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table C-5: Results from loading conditions of Hilo office building interior column

Moment	Axial Load	Shear @ d	Shear @ d + h_c	Load Combination
k-ft	Kips	Kips	Kips	
Floor 1				
134	811.8	43	26	1.2D+Ftsu+0.5L (Hydro)
134	566.1	43	26	0.9D+Ftsu (Hydro)
Floor 2				
108	676.5	32	15	1.2D+Ftsu+0.5L (Hydro)
108	471.75	32	15	0.9D+Ftsu (Hydro)
Floor 3				
105	541.2	36	20	1.2D+Ftsu+0.5L (Hydro)
105	377.4	36	20	0.9D+Ftsu (Hydro)
Floor 4				
48	405.9	5	5	1.2D+Ftsu+0.5L (Hydro)
48	283.05	5	5	0.9D+Ftsu (Hydro)
Floor 5				
12	270.6	1	1	1.2D+Ftsu+0.5L (Hydro)
12	188.7	1	1	0.9D+Ftsu (Hydro)
Floor 6				
3	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
3	94.35	0	0	0.9D+Ftsu (Hydro)

C.11.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure C-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure C-72 shows the interaction diagram for a typical interior column with the tsunami load combinations.

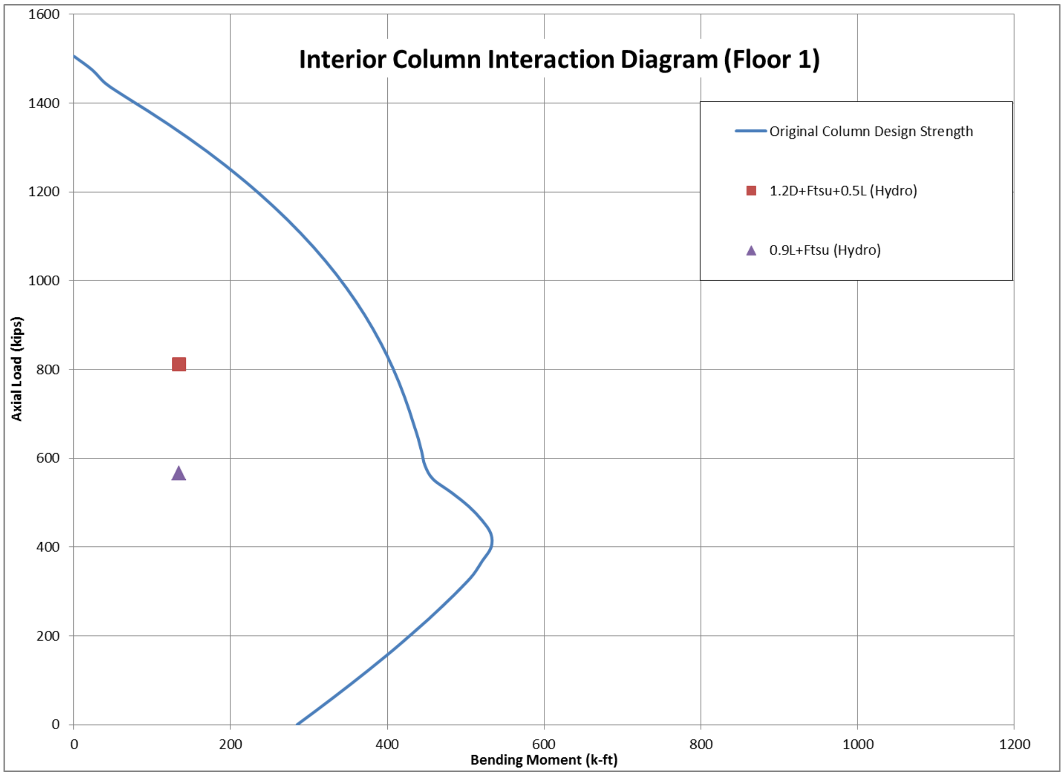


Figure C-72: Interaction diagram for typical ground floor office interior column showing tsunami load combinations

The existing interior column is therefore adequate at the first floor level, and by inspection the remaining columns are also adequate to resist the tsunami bending moments.

C.11.2.3 Interior Column Shear Design

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of (1.2D + F_{TSU} + 0.5L) per **Eqn. 6.8.3.3.-1a,** P_u = 811.8 kips.

The shear capacities of the existing 24”x24” column with 3 leg #4 Stirrups @ 4” o.c. in the end sections and 3 leg #3 Stirrups @ 5” o.c. in the center section are given by:

$\varnothing V_n = \varnothing(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{811{,}800}{2{,}000\times 24\times 24}\right)24\times 21.5/1{,}000$ = 111 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.2)\times 60{,}000\times 21.5}{4\times 1{,}000}$ = 194 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.11)\times 60{,}000\times 21.5}{5\times 1{,}000}$ = 85 kips

Therefore in the end sections, $\varnothing V_n$ = 0.75 (111 + 194) = 229 k

and in the center sections, $\varnothing V_n$ = 0.75 (111 + 85) = 147 k

At d: V_u = 43 k < ϕV_n = 229 k, therefore the column is adequate for shear at the edge

At d + h_c: V_u = 26 k < ϕV_n = 147 k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear forces.

C.12 Tsunami Design for Residential Building

C.12.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

*In lieu of performing detailed hydrostatic and hydrodynamic analysis, **Eqn. 6.10.1-1** provides a simplified but conservative estimate of the maximum lateral load on the building.*

$$f_{uw} = 2.5 I_{tsu} \gamma_s h_{max}^2 = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 55.12^2 = 535\ kip/ft$$

Assuming $C_{cx} = 0.7$ *controls per* ***Section 6.8.7***

Therefore $F = 0.7 \times 254 \times 535 = 95{,}074\ kips$

This lateral load can be compared with 0.75 Ω_o E_h = 0.75 x 2.5 x 2,435 = 4,566 kips < 95,074 kips. Therefore the LFRS is not adequate to satisfy this requirement and the detailed analysis for LC2 and LC3 shown below is recommended. The components can also be designed on the basis of this conservative uniform distributed force with the appropriate width b dimensions (but that is not illustrated here).

C.12.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

C.12.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} ≤ h_{max} = 55.12 ft

≤ 14 ft (ground floor story height)

≤ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

Load Case 1 also requires application of the associated hydrodynamic drag on the entire building. However this will not control since buoyancy need not be considered.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2} \rho_s I_{tsu} C_d C_{cx} B (hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4575 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254' overall width of building

h = 8'

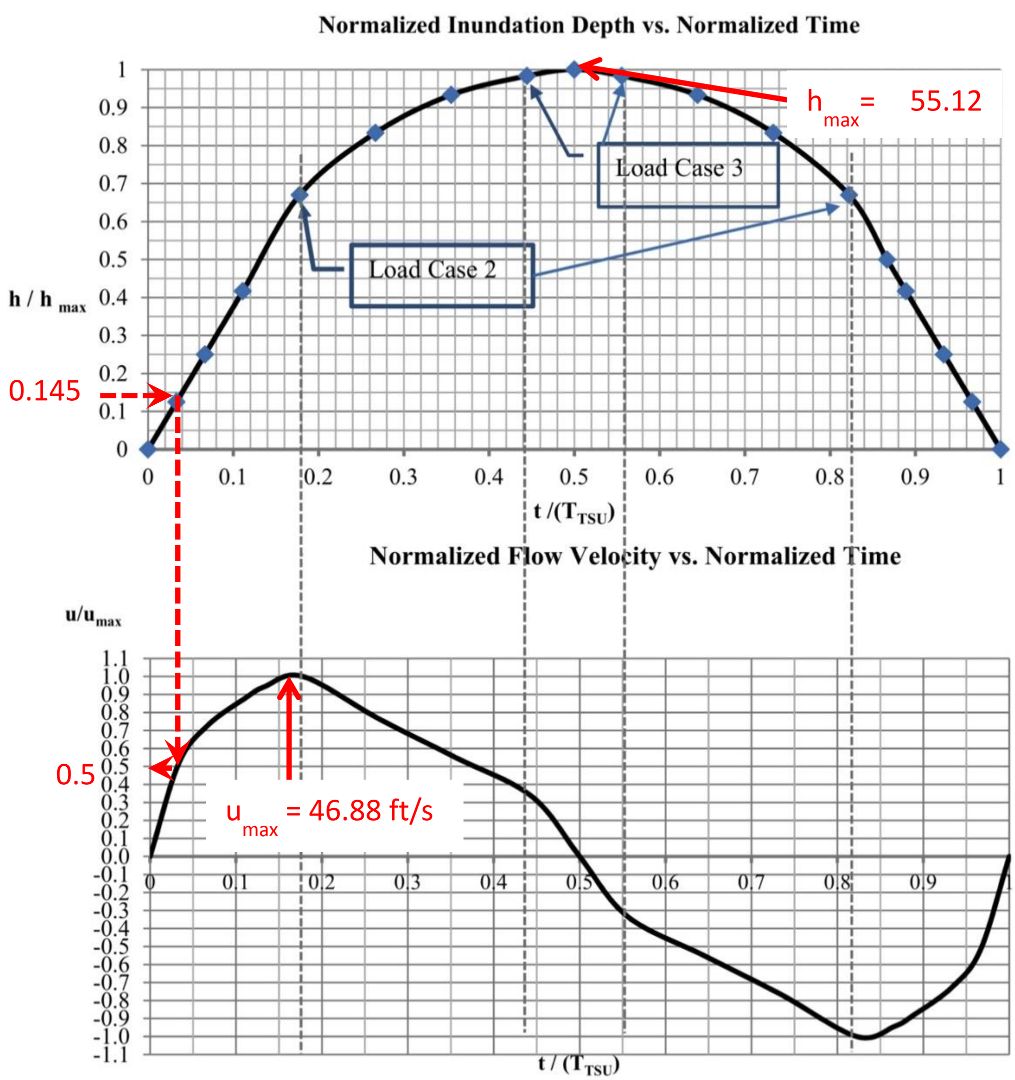


Figure C-73: Determining "u" for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/55.12 = 0.145. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.038. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.4. Therefore the flow velocity is u = 0.5 x 46.88 = 23.44 fps.

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 23.44^2)/1000 = 1{,}790\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail to relieve lateral load on the structural frame. Note that a portion of this load will go to the ground floor slab, which reduces the load that has to be resisted by the lateral force resisting system. The entire lateral load must be resisted by the deep foundations assuming maximum scour has already occurred.

C.12.2.2 Load Case 2: Maximum Flow Velocity

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is 2/3h_{max} = 2/3 x 55.12 = 36.75 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/36.75= 6.9)

Since the inundation depth of 36.75 feet exceeds the bottom of the fourth floor slab (12'+9'+9' – 8"/12) = 29.33', the inundated area of the beams must be included in the closure coefficient, which is determined as follows:

h_{sx} = 36.75 ft.

A_{col} = 32 x 1.67' x (36.75'-3 x 0.67') = 1853 ft^2

A_{wall} = 2 x 28' x (36.75'-3 x 0.67') + 2 x 10' x (36.75'-3 x 0.67') = 2641 ft^2

A_{Beam} = A_{Slab} = 3 x 254' x 0.67' = 508 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5\times A_{beam}}{Bh_{sx}} = \frac{\sum((1853+2641)+1.5\times 508}{254\times 36.75} = 0.563 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

$u = u_{max}$ = 46.88 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(36.75 \times 46.88^2)/1000 = 19{,}744\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 36.75 feet above grade. The lateral force resisting system for the structure must be evaluated for this load. During drawdown the same pressure needs to be applied to the inland elevation and the lateral force resisting system evaluated for this load.

C.12.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 55.12 ft. and the flow velocity is $1/3u_{max}$ = 1/3 x 46.88 = 15.63 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/55.12= 4.6)

Since the inundation depth of 55.12 ft exceeds the sixth floor slab elevation of 48 ft, the closure coefficient is determined as follows:

h_{sx} = 55.12 ft.

A_{col} = 32 x 1.67' x (55.12'-5 x 0.67') = 2762 ft^2

A_{wall} = 2 x 28' x (55.12'-5 x 0.67') + 2 x 10' x (55.12'-5 x 0.67') = 3936 ft^2

$A_{Beam} = A_{Slab}$ = 5 x 254' x 0.67' = 847ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum(2762+3936)+1.5\times 847}{254' \times 55.12'} = 0.569 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 15.63 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(55.12 \times 15.63^2)/1000 = 3{,}291\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 55.12 feet above grade. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth.

C.12.2.4 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category D, **Section 6.8.3.4** permits the use of $0.75\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear. From the seismic design of this structure, E_h = 2,435 kips. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 2.5 \times 2{,}435 = 4{,}566\ kips$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 19,744 kips applied over a height of 36.75 ft. A portion of this load will be resisted by the grade beam/foundation system, reducing the overall load by 3,224 kips. (**Figure C-74**)

$$0.75\Omega_o E_h = 4{,}566\ kips < 16{,}520\ kips$$

Therefor the lateral force resisting system is not adequate and the ETABS model needs to be revaluated. This ETABS analysis resulted in the column forces shown in **Figure C-76** - **Figure C-79** for floors one through four, respectively. These systemic loads on each element of the LFRS must be combined with the component loads on that member.

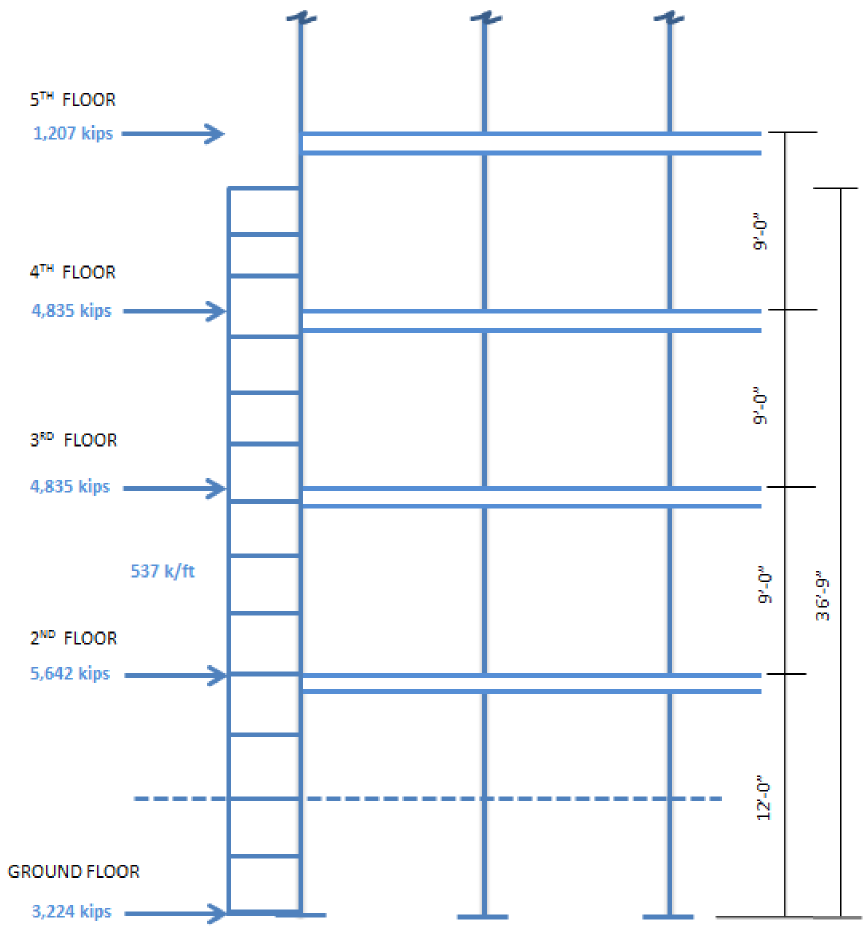


Figure C-74: LC2 Tsunami loads on overall Hilo Residential building

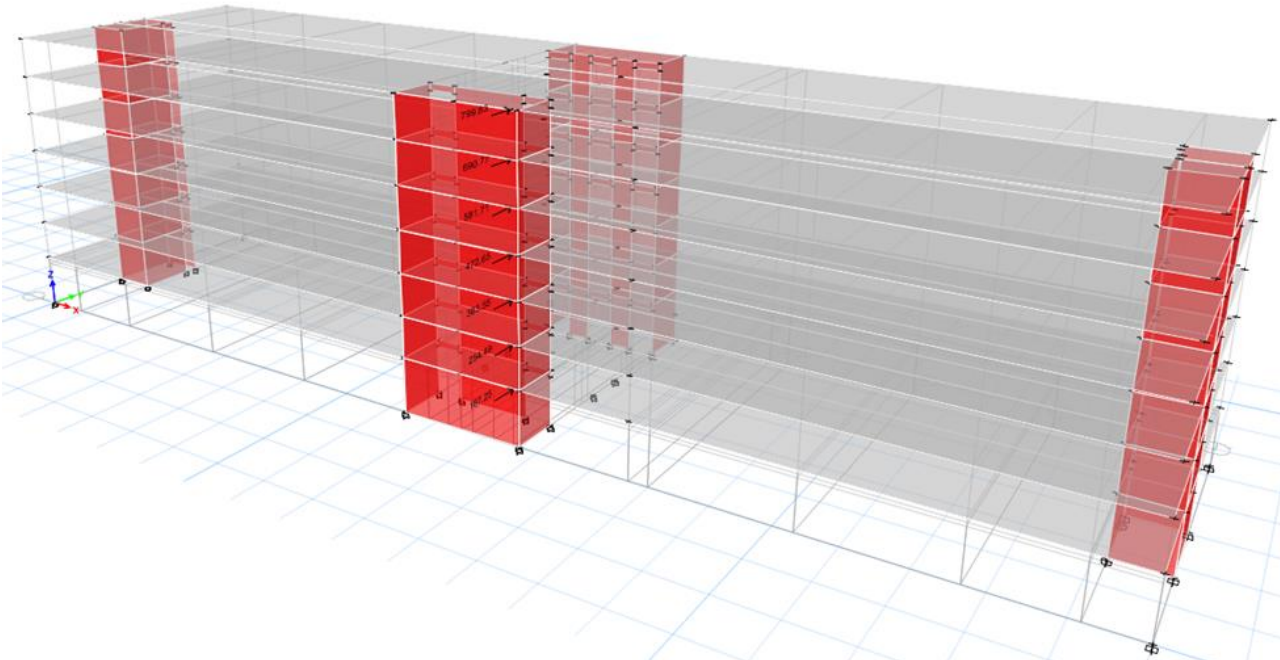


Figure C-75: ETABS computer model of residential building subjected to elevated seismic loads to meet the tsunami demand at the Hilo location

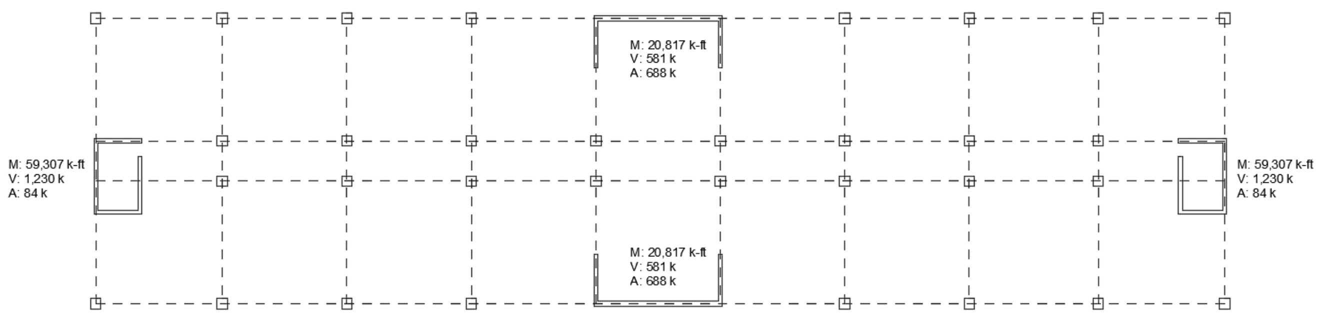


Figure C-76: ETABS output of axial load, shear force and bending moment at the base of each structural wall ground floor

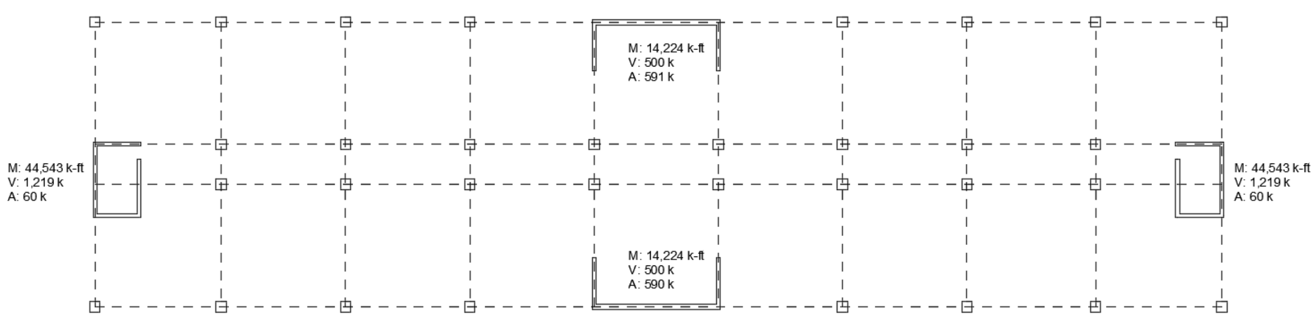


Figure C-77: ETABS output of axial load, shear force and bending moment at the base of each structural wall 2nd floor

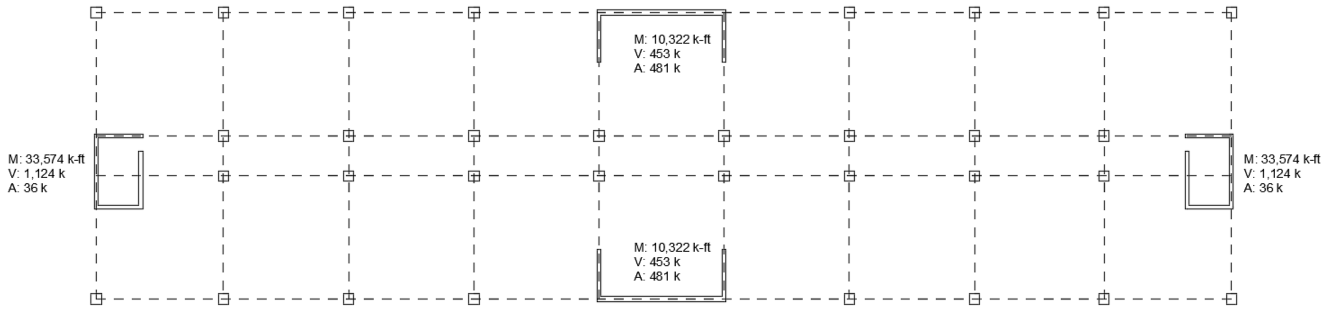


Figure C-78: ETABS output of axial load, shear force and bending moment at the base of each structural wall 3rd floor

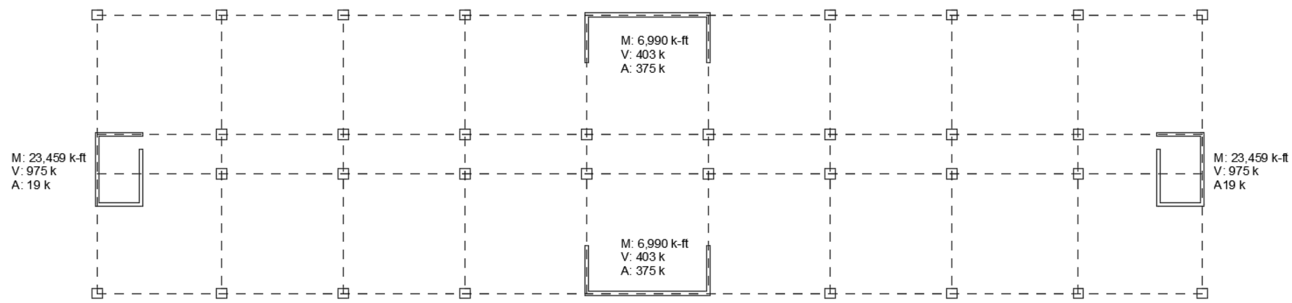


Figure C-79: ETABS output of axial load, shear force and bending moment at the base of each structural wall 4th floor

C.13 Component Design

C.13.1 Drag Force on Components - Section 6.10.2.2

C.13.1.1 Exterior Columns

Exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** requires that C_d = 2.0 and the width dimension, b, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Therefore b = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 36.75 ft and u_{max} = 46.88 fps.

The hydrodynamic drag is computed using **Eqn 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(36.75 \times 4688^2)/1000 = 3{,}482\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 3482/36.75 = 94.77 kips/ft over the lower 36.75 feet of the column. The column must be designed for this load combined with gravity loads per **Section 6.8.3.3**.

C.13.1.2 Interior Columns

Interior columns are 20" (1.67 ft) square R.C. columns. The controlling load case will be LC2, when the inundation depth is is h_e = 36.75 ft and u_{max} = 46.88 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**) and b = 1.67 ft since no debris accumulation is considered for interior column.

Therefore $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 1.67(36.75 \times 46.88^2)/1000 = 296\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 296/36.75 = 8.05 kips/ft over the lower 36.75 feet of the column. This load must be combined with gravity loads per **Section 6.8.3.3** and the column capacity verified.

C.13.2 Tsunami Loads on Structural Walls, F_w – Section 6.10.2.3

Since tsunami bores are anticipated at this location, the lateral load on the structural walls is given by **Eqn. 6.10-5a** or **Eqn. 6.10-5b**, depending on the flow depth relative to the wall width:

Eqn. 6.10-5a $F_W = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$

Eqn. 6.10-5b $F_W = \frac{3}{4}\rho_s I_{tsu} C_d b(h_e u^2)$ when $\frac{b_w}{h_e} \geq 3$

Where C_d = 2.0 for a wall per **Table 6.10-2**, and

Elevator Walls:

b = 28' for the elevator walls

Elevator $\frac{b_w}{h_e} = \frac{28'}{36.75'} = 0.76 \ngeq 3 \therefore \text{Eqn. 6.10-5a}$

The controlling load case will be LC2, where h_e = 36.75 ft and u = 46.88 fps.

Therefore, for the 28' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 28(36.75 \times 46.88^2)/1000 = 4{,}975\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 4,975/(28x36.75) = 4,835 psf over the lower 36.75 ft of the walls. ← **(CONTROLS)**

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3} \rightarrow h_e \geq \frac{b_w}{3'} = \frac{28'}{3'}$ = 9.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 9.33 ft, the flow velocity can be obtained from **ASCE**

7-16 **Figure 6.8-1**, as shown in **Figure C-80**. The resulting velocity is h/h_{max} = 9.33'/55.12' = 0.169. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.4. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.6. Therefore the flow velocity is u = 0.6 x 46.88 = 28.1 fps. The bore loading is computed for h_e = 9.33 ft and u = 28.1 fps.

Therefore for the 28' wide elevator wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 28(9.33 \times 28.1^2)/1000 = 681\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 681/(28x9.33) = 2,607 psf over the lower 9.33 ft of the walls. This will not govern when compared with the pressure from hydrodynamic drag in **Eqn. 6.10-5a**.

Stairwell Walls:

b = 10' for the elevator walls

Stairwell $\frac{b_w}{h_e} = \frac{10'}{36.75'} = 0.27 \ngeq 3 \therefore \text{Eqn. 6.10-5a}$

The controlling load case will be LC2, where h_e = 36.75 ft and u = 46.88 fps.

Therefore, for the 10' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 10(36.75 \times 46.88^2)/1000 = 1{,}777\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 1,777/(10x36.75) = 4,835 psf over the lower 36.75 ft of the walls. ← **(CONTROLS)**

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3} \rightarrow h_e \geq \frac{b_w}{3'} = \frac{10'}{3'}$ = 3.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 3.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure C-80**. The resulting velocity is h/h_{max} = 3.33'/55.12' = 0.06. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.02. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.3. Therefore the flow velocity is u = 0.3 x 46.88 = 14.1 fps. The bore loading is computed for h_e = 3.33 ft and u = 14.1 fps.

Therefore for the 10' wide stairwell wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 10(3.33 \times 14.1^2)/1000 = 21.8 kips$

These loads are applied to the walls as a uniformly distributed pressure of 21.8/(10x3.33) = 654 psf over the lower 3.33 ft of the walls. This will not govern when compared with the pressure from hydrodynamic drag in Eqn. **6.10-5a**.

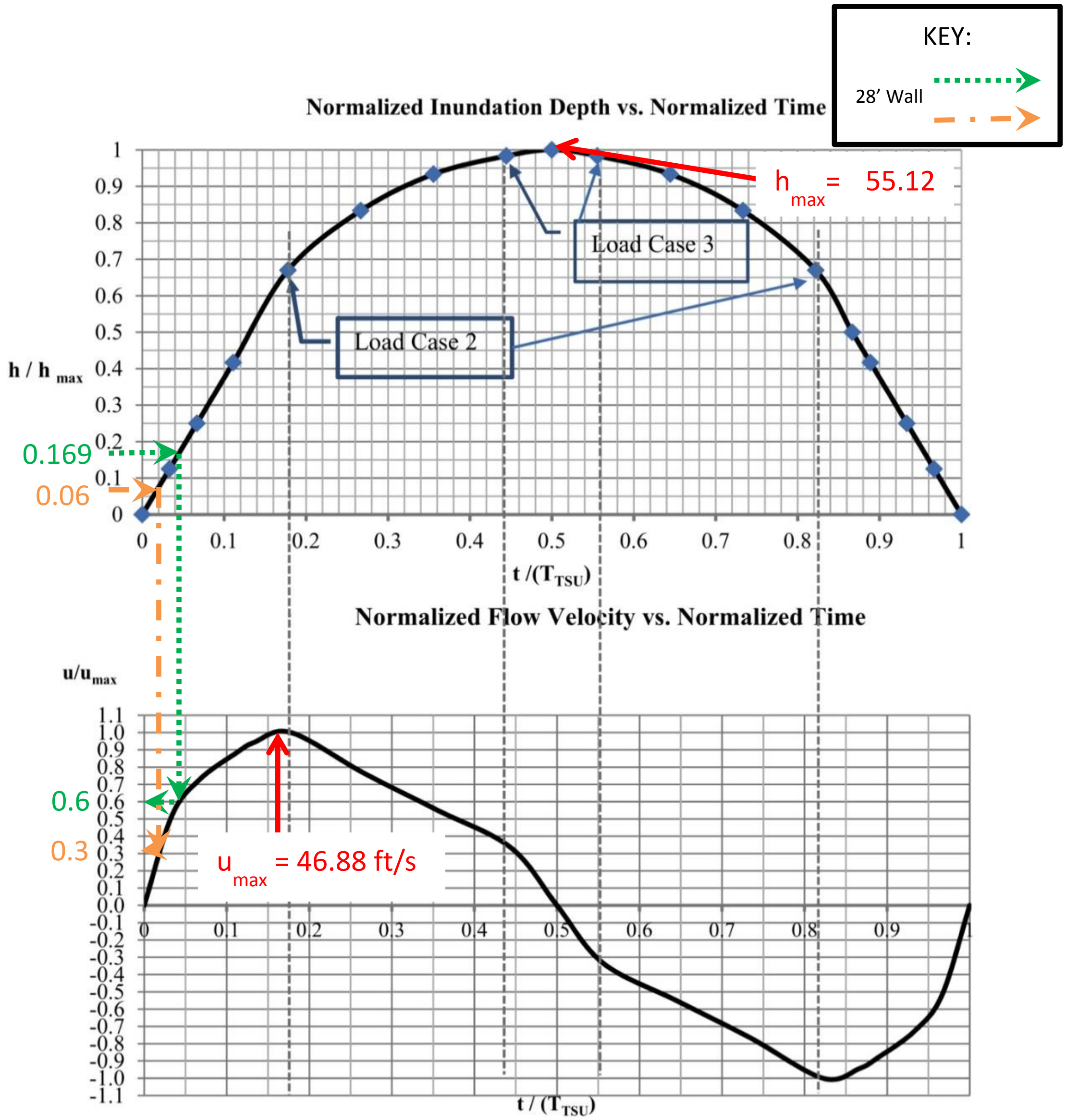


Figure C-80: Determining "u" for Eqn. 6.10-5b with ASCE 7-16 Figure 6.8-1

C.13.3 Hydrodynamic Pressures associated with Slabs – Section 6.10.3

C.13.3.1 Flow Stagnation Pressure – Section 6.10.3.1

The Mechanical/Electrical room on Gridline D between Gridlines 5 and 6 is enclosed on all sides by structural walls. Tsunami flow entering through the two door openings will result in flow stagnation pressurization of this room, given by **Eqn. 6.10-8** as:

$$P_p = \frac{1}{2}\rho_s I_{tsu} u^2$$

Assuming that the door openings are 7 ft high, the stagnation pressurization is based on the maximum flow velocity occurring at this or greater depths, ie. when the door opening is fully submerged. The flow

velocity will therefore be the maximum of 46.88 fps which occurs when the flow depth is 36.75 ft (**Figure 6.8-1**, LC2). Therefore;

$$P_p = \frac{1}{2} \times 2.2 \times 1.0 \times 46.88^2 = 2{,}418\ psf$$

The structural walls surrounding this room must be evaluated for an outward pressure of 2,418 psf, in combination with gravity loads per **Section 6.8.3.3**. The floor slab above this room must be designed for a net uplift pressure given by $0.9D + F_{TSU}$ = -0.9x100 + 2,418 = 2,328 psf upwards. This will require additional top reinforcement in this slab and shear reinforcement around the slab perimeter. In order to reduce the amount of additional reinforcement, one could perform a non-linear analysis of the floor slab following the provisions of ASCE 41. A simpler alternative may be to design the floor slab in the mechanical room as a breakaway slab, as shown in **Figure C-81,** in order to relieve pressure. This will apply to all levels up to h_{max}

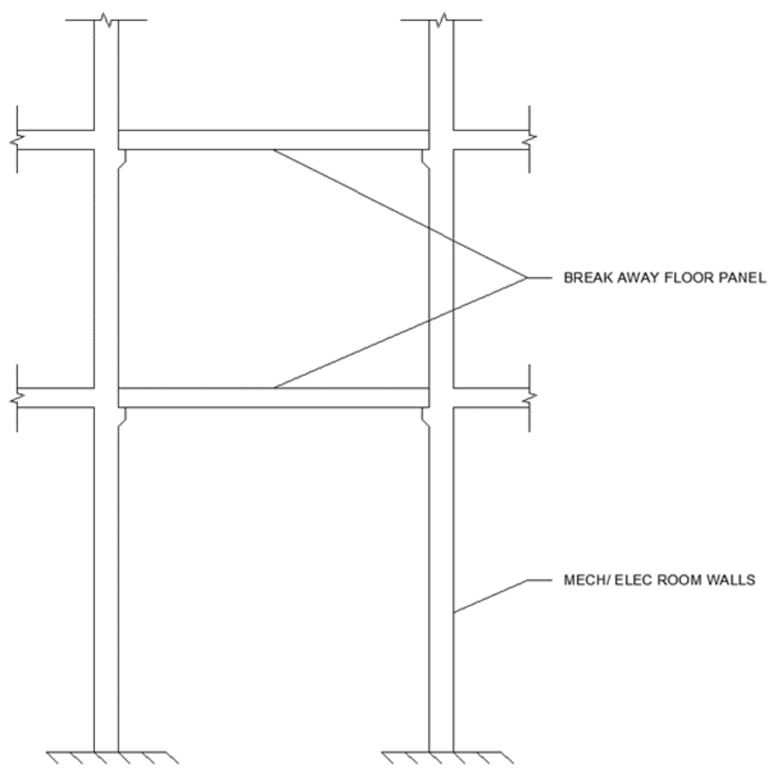


Figure C-81: Mechanical/Electrical room break-away floor panels applied to all levels up to h_{max}

C.13.3.2 Hydrodynamic Surge Uplift at Horizontal Slabs – Section 6.10.3.2

If slabs are submerged during the tsunami, they must be designed for uplift, with a specified minimum of 20 psf (**Section 6.10.3.2.1**). The uplift may increase if the ground floor is sloped, causing an upward component of flow velocity (**Section 6.10.3.2.2**). This is not the case for this building.

The resulting minimum uplift of 20 psf is much smaller than the dead weight of the slab (100 psf), therefore this uplift will not affect the slab design.

C.13.3.3 Tsunami Bore Flow Entrapped in Structural Wall-Slab Recesses – Section 6.10.3.3

If a tsunami bore is entrapped in a structural wall-slab recess, then large pressures can develop on the slab and wall (**Section 6.10.3.3.1**). Although tsunami bores are anticipated at this location, the flow can pass freely around the wall elements in this building. Therefor this condition does not apply.

C.14 Debris Impact Loads - Section 6.11

The inundation depth exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads.

C.14.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330 I_{tsu} C_o = 330 \times 1.0 \times .65 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force can be reduced to 50%, or 107.25 kips. This load must be applied to the 20" square exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with other tsunami loads and it need not be applied to interior columns.

This equivalent static impact load of 107.25 kips must also be applied to any structural walls on the perimeter of the building. This applies to the 28 ft wide elevator walls on both exterior sides of the building (GLs A and D) since impact must be considered during inflow and outflow conditions. Evaluation of the wall capacity is based on a tributary wall width of half the wall height. Since the wall unbraced height is (12' – 8"/12) = 11.33', the tributary width is 5.67 ft.

C.15 Column Design for Tsunami Loads

C.15.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 from **Figure C-17**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure C-82** and **Figure C-83** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the end of the column, where *d* = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "$d + h_e$" from the end of the column, where $d + h_e = 17.5 + 20 = 37.5$ in.

Floor 1 – 7

End Section (A)

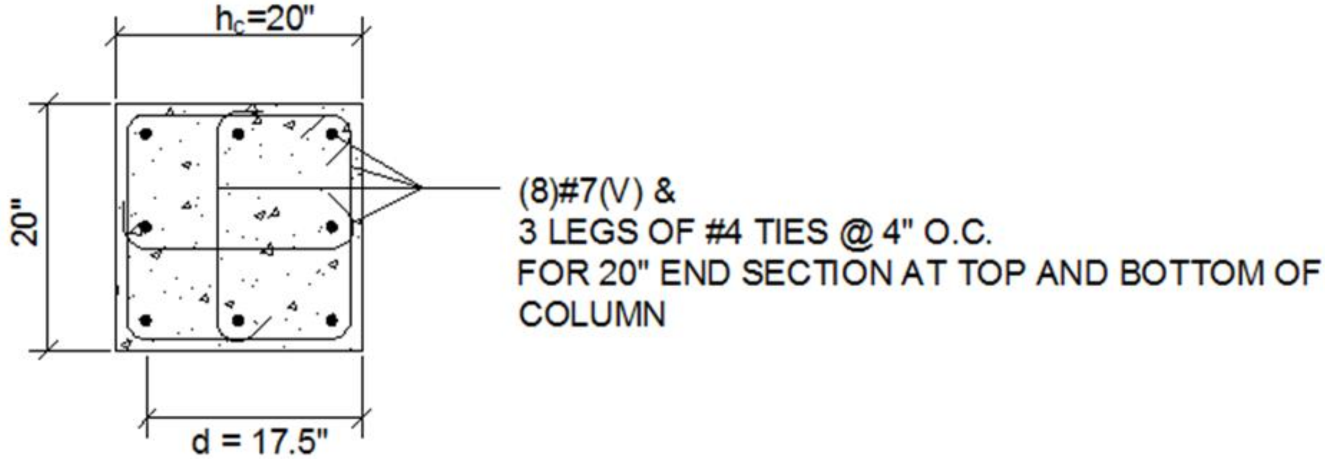


Figure C-82: Exterior column, cross-section at end of column at all floor levels based on SDC D design.

Center Section (B)

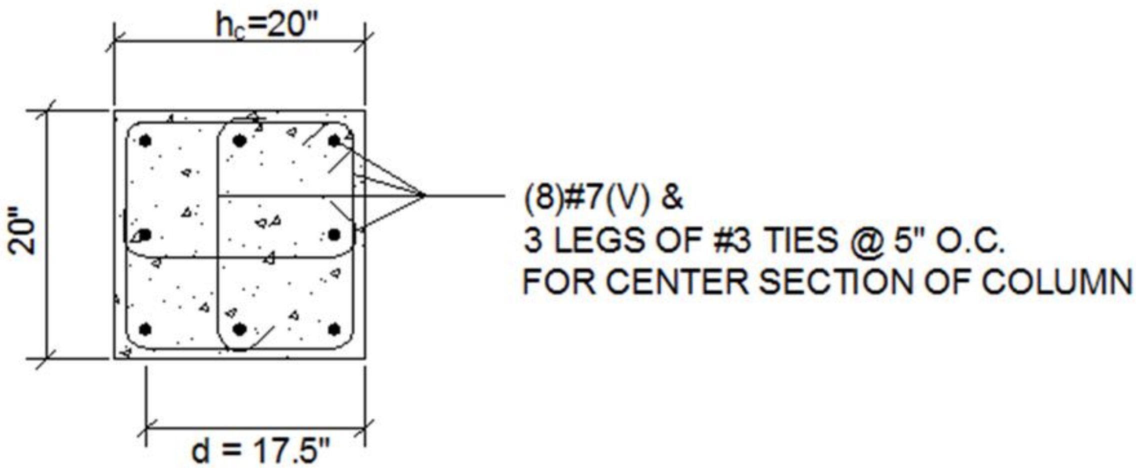


Figure C-83: Exterior column, cross-section at center of column at all floor levels based on SDC D design.

C.15.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 14.58 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

$$P_D = [128(14.58)(28)(6)+ 123(14.58)(28)+ 90(28)(6)+1.67^2(150)(66)]/1000 = 406\ k$$

Floor Live load reduction factor = $0.25+15/[4(14.58)(28)(5)]^{0.5} = 0.402$*, therefore, column base live load is:*

$$P_L = 0.402[55(14.58)(28)(6)]/1000 = 54.2\ k$$

Roof Live Load reduction factor = $R_1R_2= [1.2-(0.001)(14.58)(28)](1.0) = 0.792$*, therefore, roof live load is:*

$P_{Lr} = 0.792(20)(14.58)(28)/1000 = 6.47kF$

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

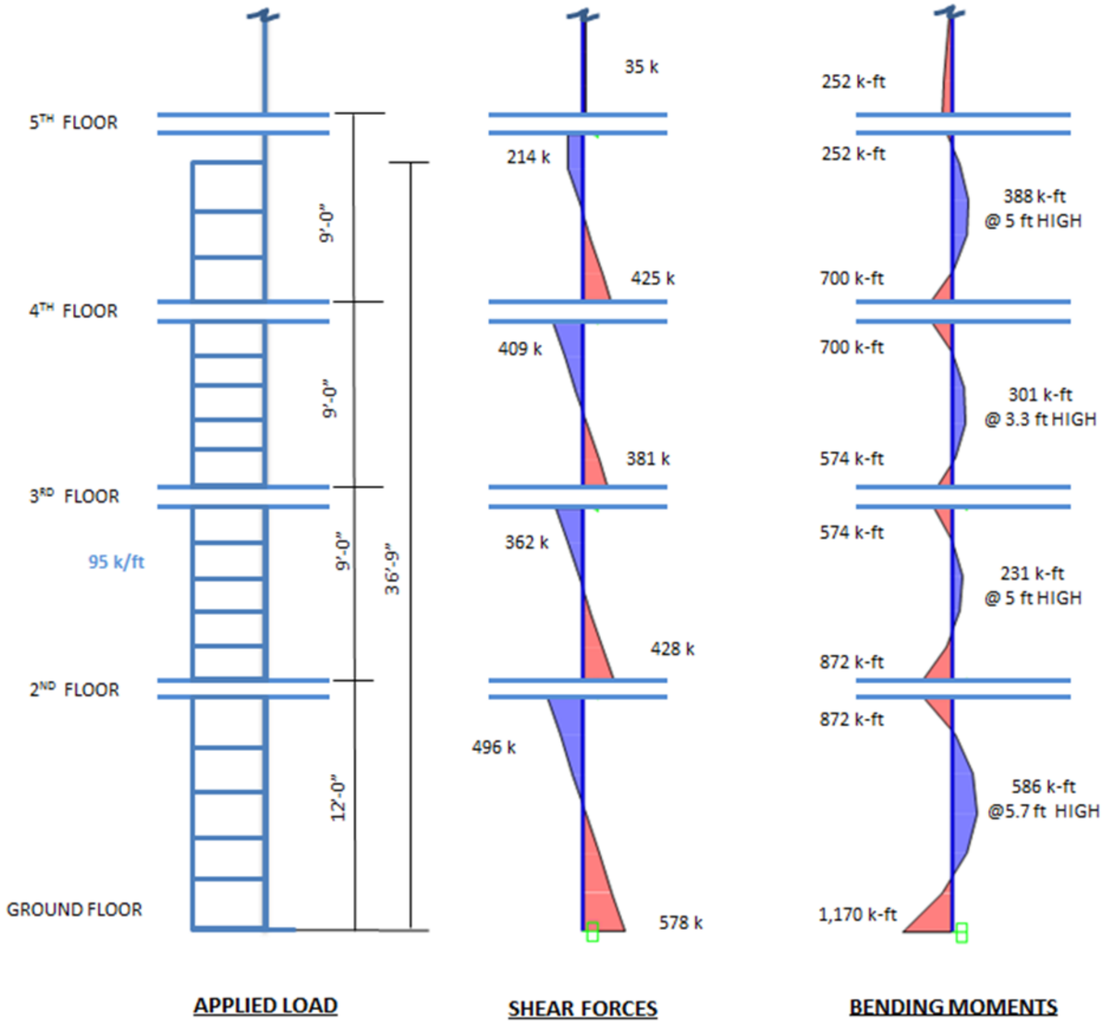


Figure C-84: Hydrodynamic loading on exterior column of Hilo residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just above and below each inundated floor slab for the maximum shear and near the mid-height of the clear column height for maximum bending moments. Samples of the resulting shear force and bending moment diagrams are provided below. Similar diagrams and similar shear and bending moments would result if the impact load was applied at the other end of each column.

Impact load at d:

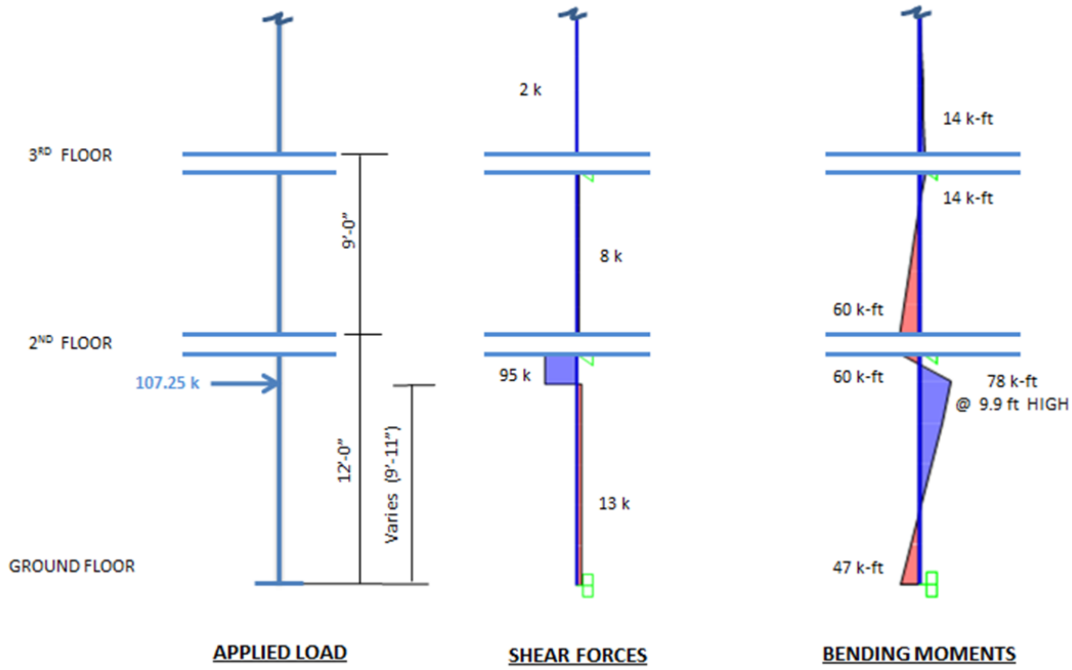


Figure C-85: Impact load applied at "d" away from the end of column on the ground floor

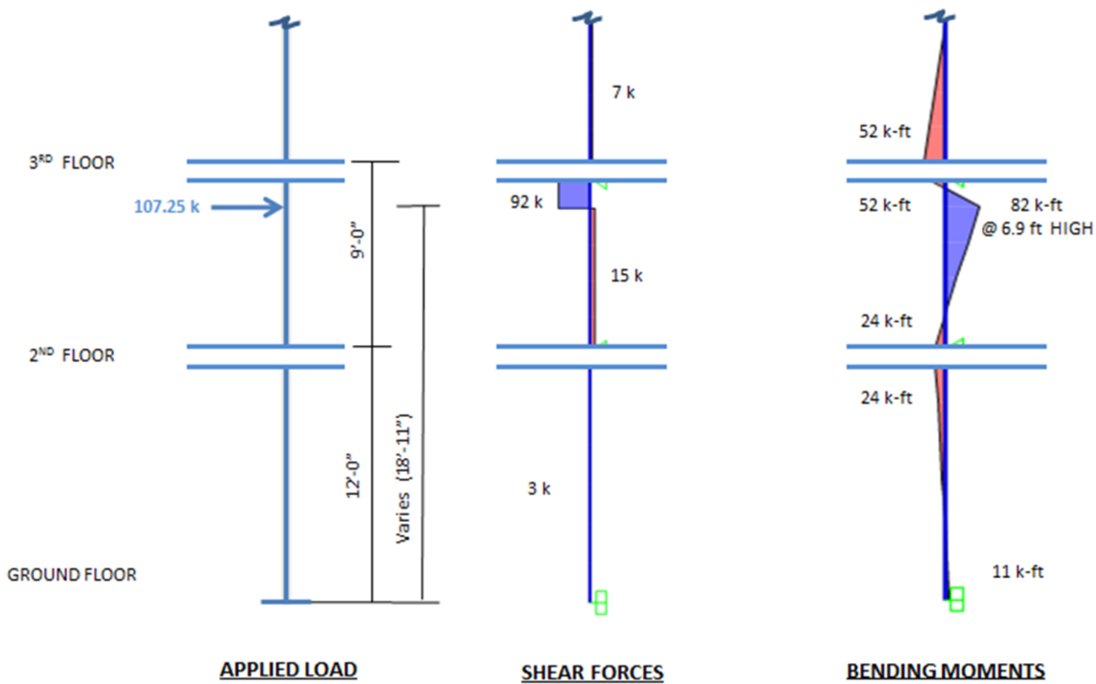


Figure C-86: Impact load applied at "d" away from the end of column on the 2nd floor

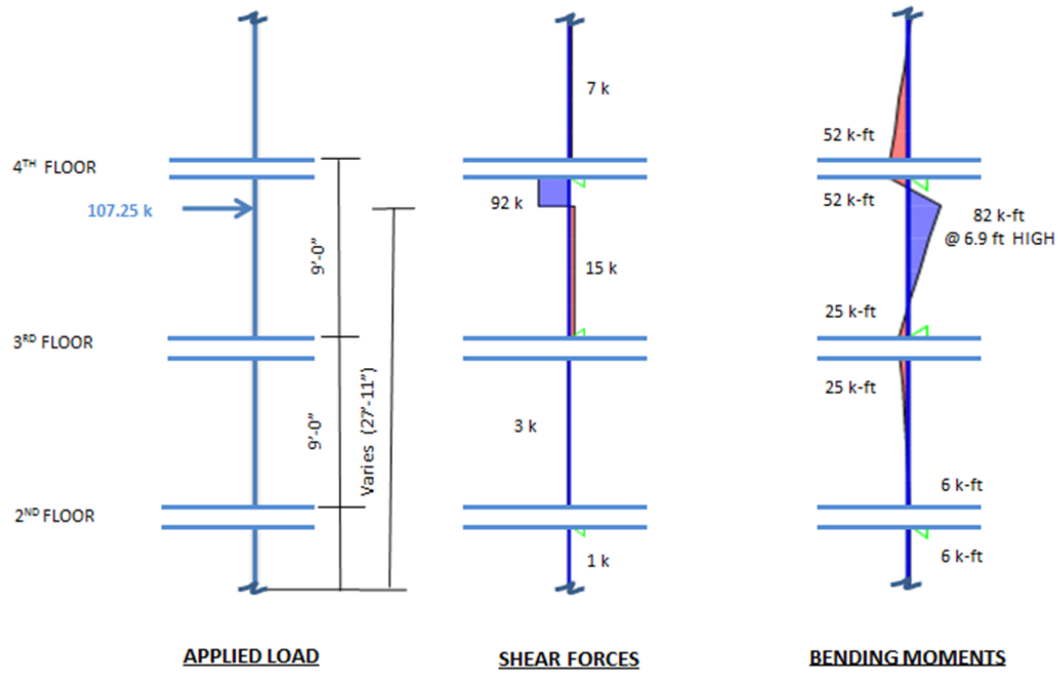


Figure C-87: Impact load applied at "d" away from the end of column on the 3rd floor

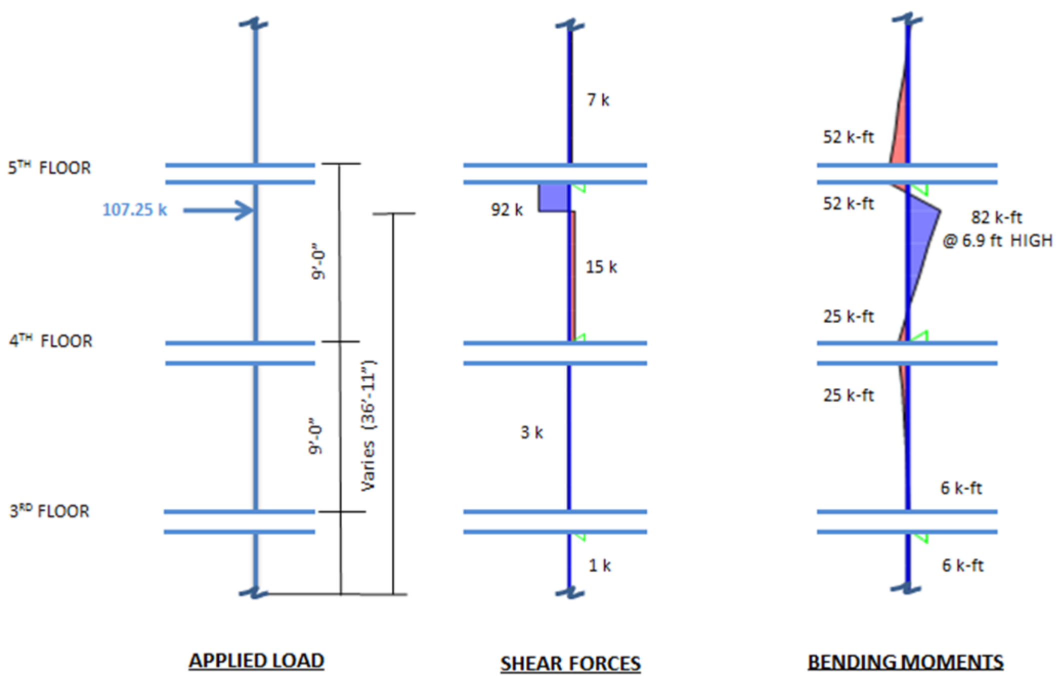


Figure C-88: Impact load applied at "d" away from the end of column on the 4th floor

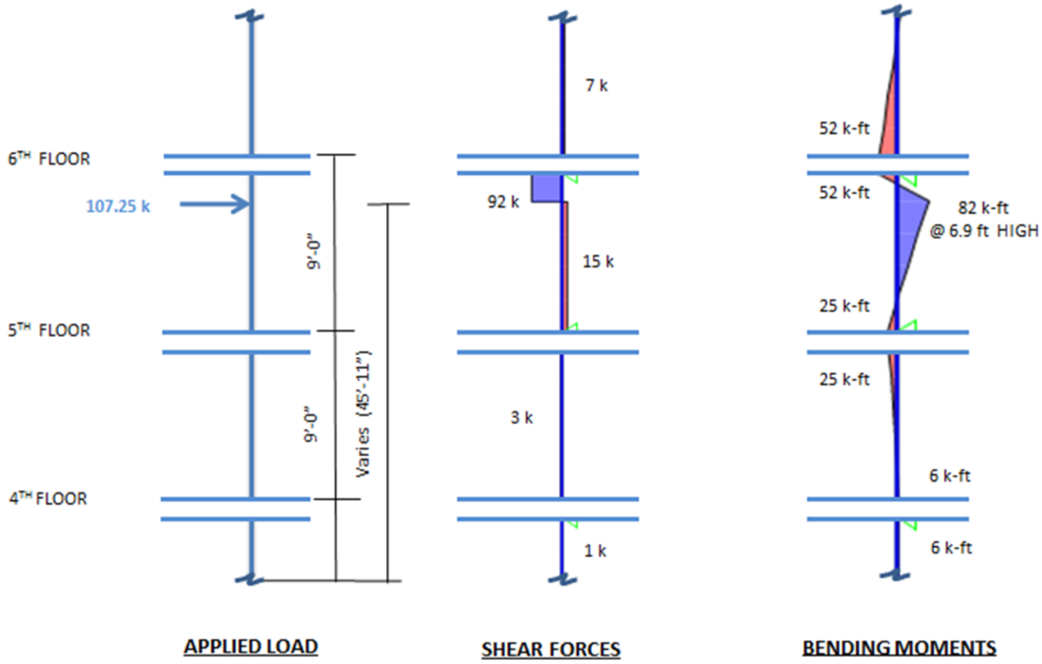


Figure C-89: Impact load applied at "d" away from the end of column on the 5th floor

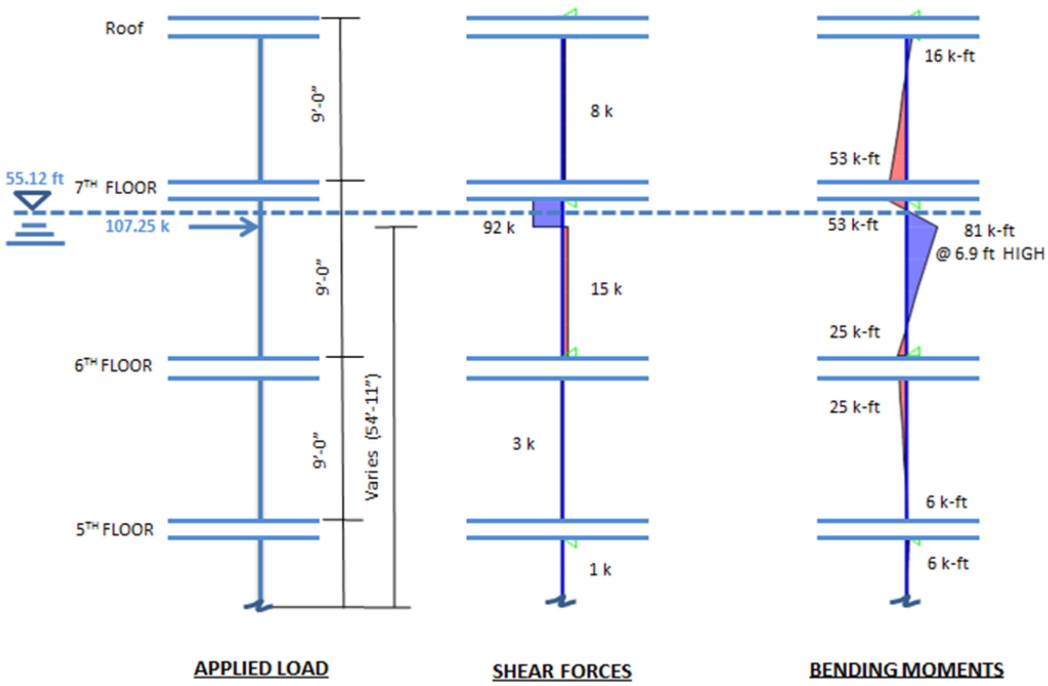


Figure C-90: Impact load applied at "d" away from the end of column on the 6th floor

Impact load at d + h_c:

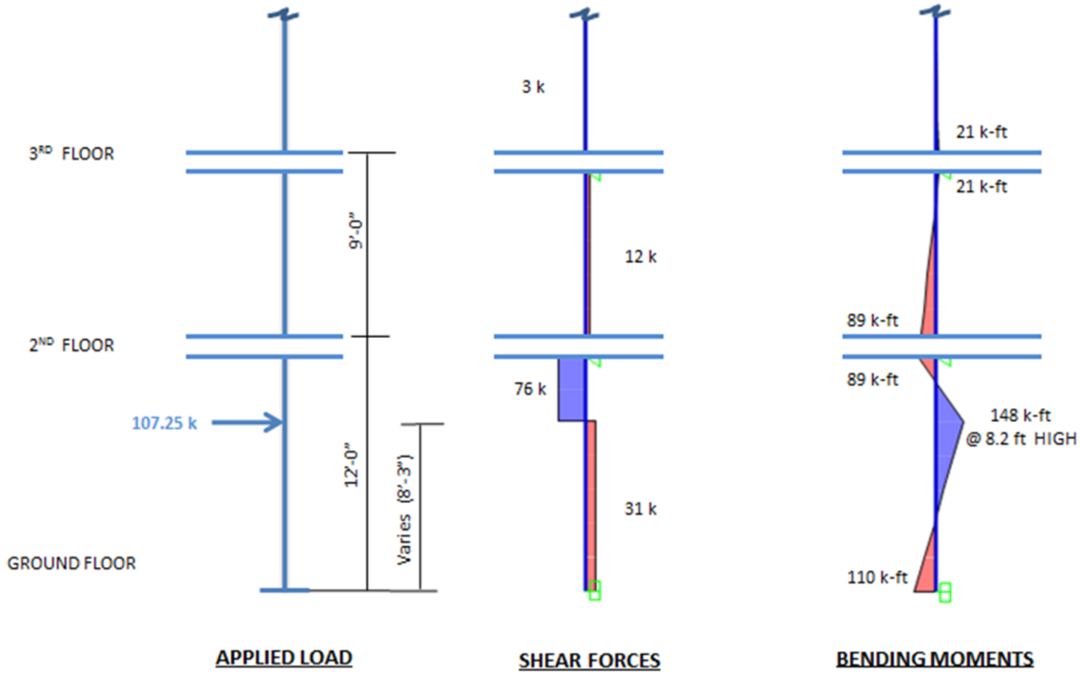


Figure C-91: Impact load applied at "d + h_c" away from the end of column on the ground floor

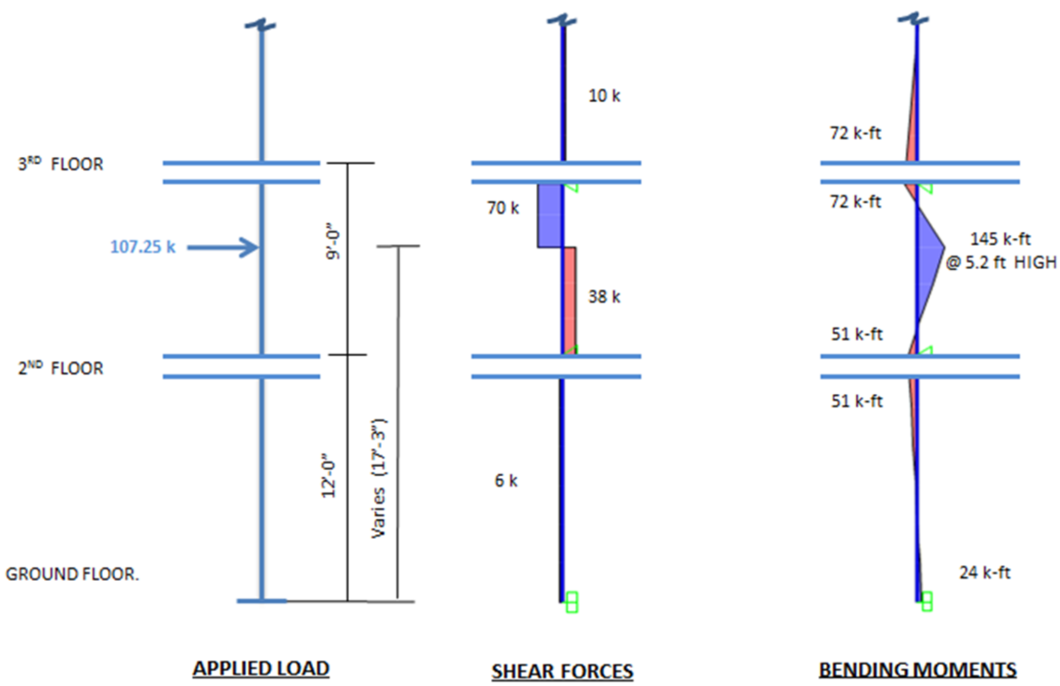


Figure C-92: Impact load applied at "d + h_c" away from the end of column on the 2nd floor

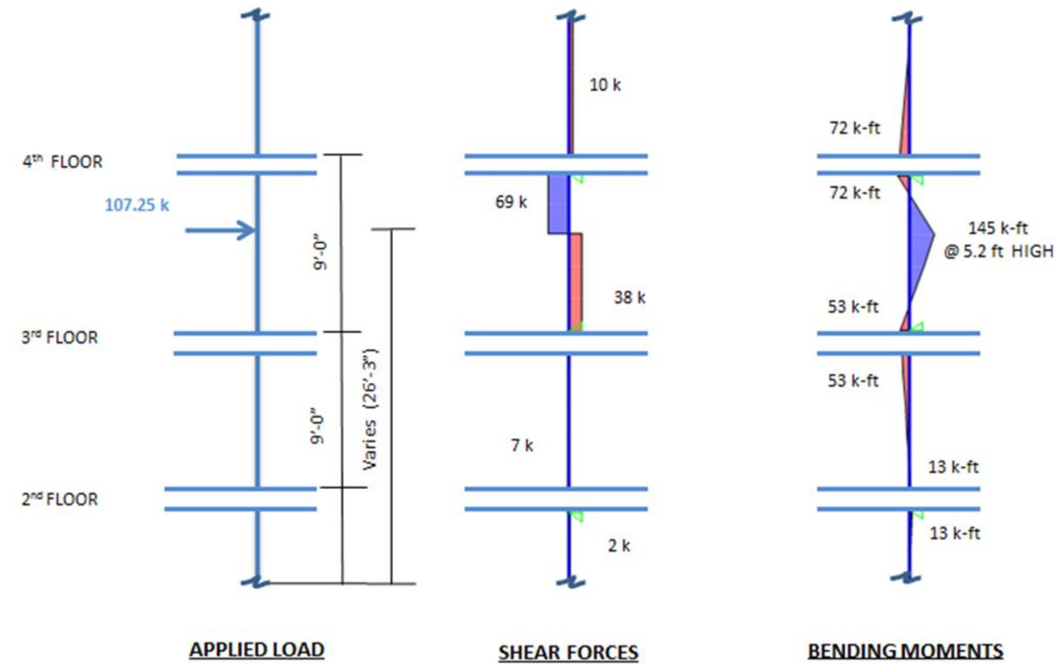


Figure C-93: Impact load applied at "d + h_c" away from the end of column on the 3rd floor

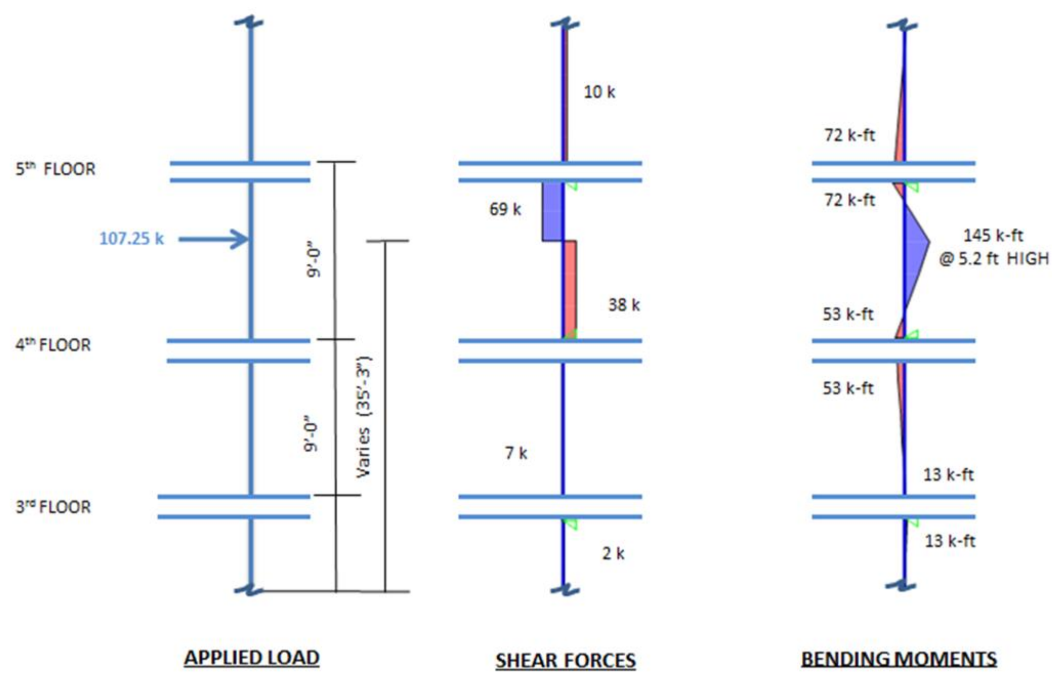


Figure C-94: Impact load applied at "d + h_c" away from the end of column on the 4th floor

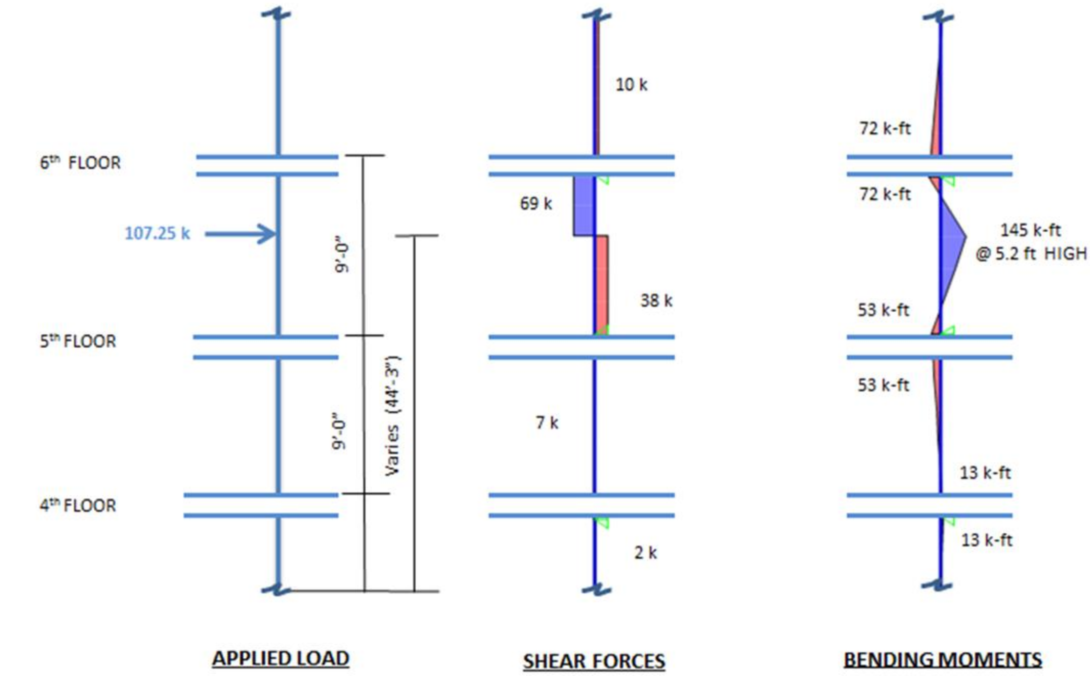


Figure C-95: Impact load applied at "d + h_c" away from the end of column on the 5th floor

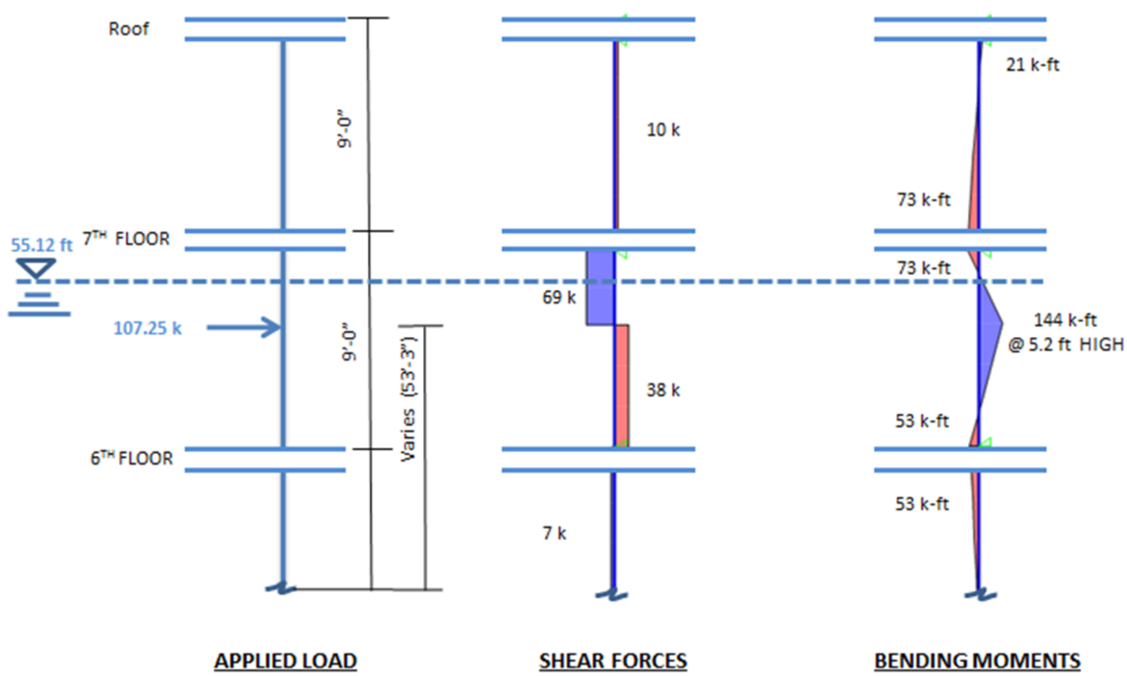


Figure C-96: Impact load applied at "d + h_c" away from the end of column on the 6th floor

Impact load at mid-height:

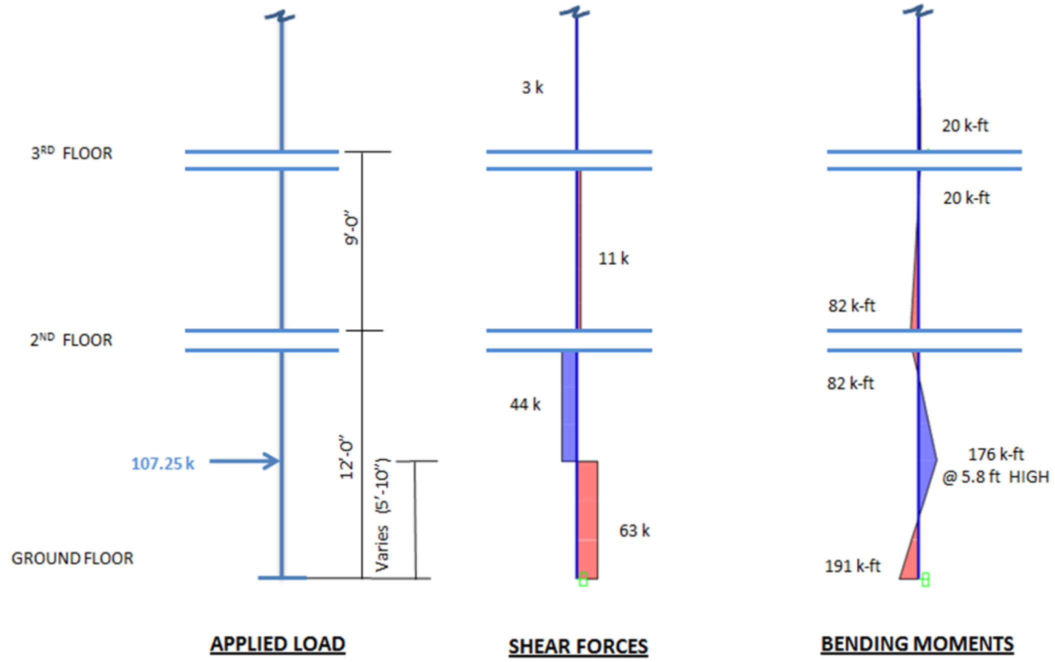


Figure C-97: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

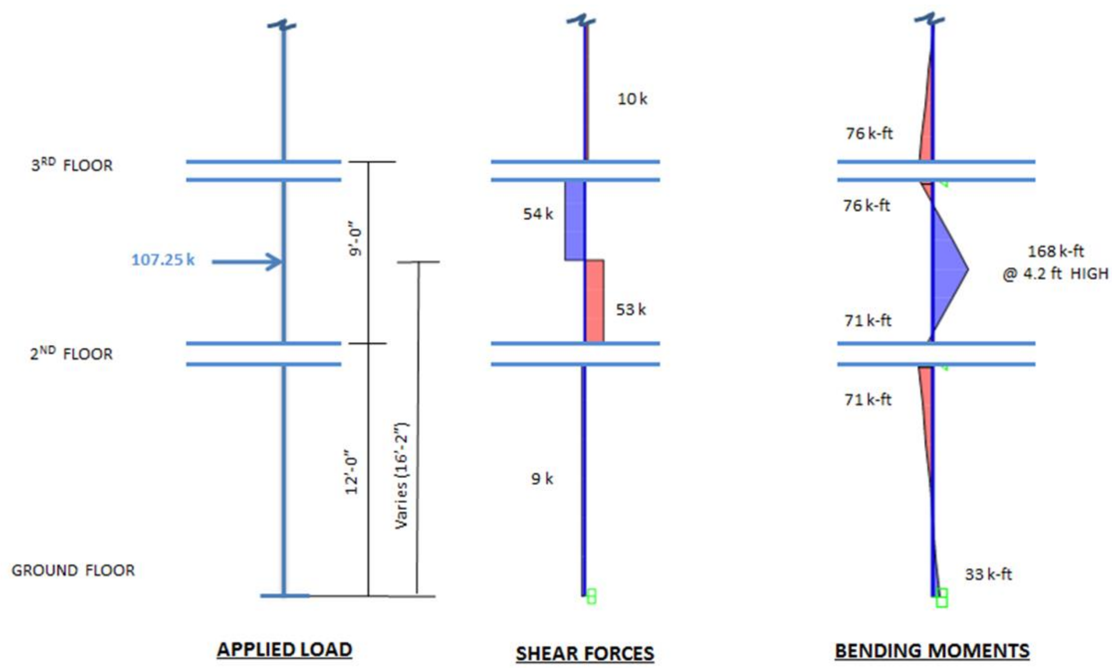


Figure C-98: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

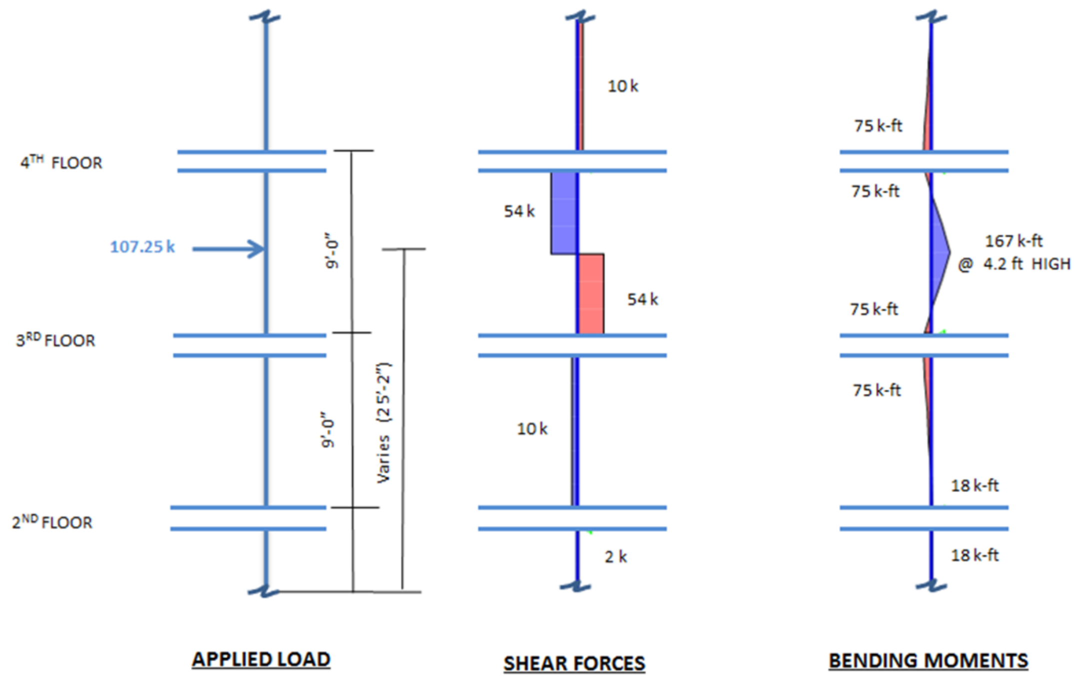


Figure C-99: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

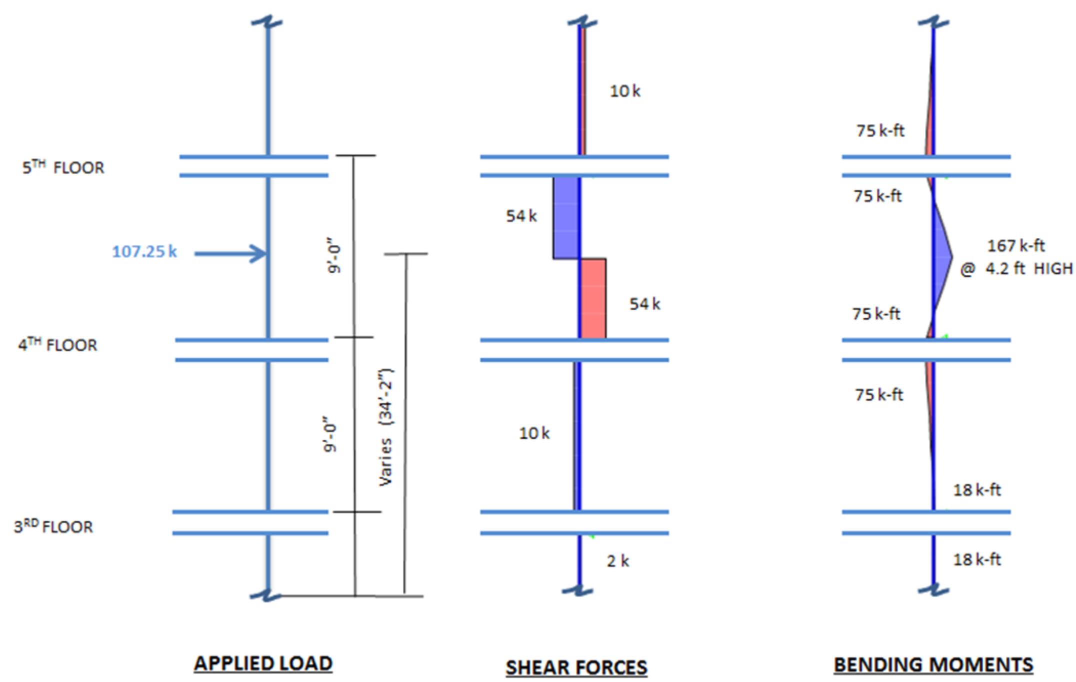


Figure C-100: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 4th floor column

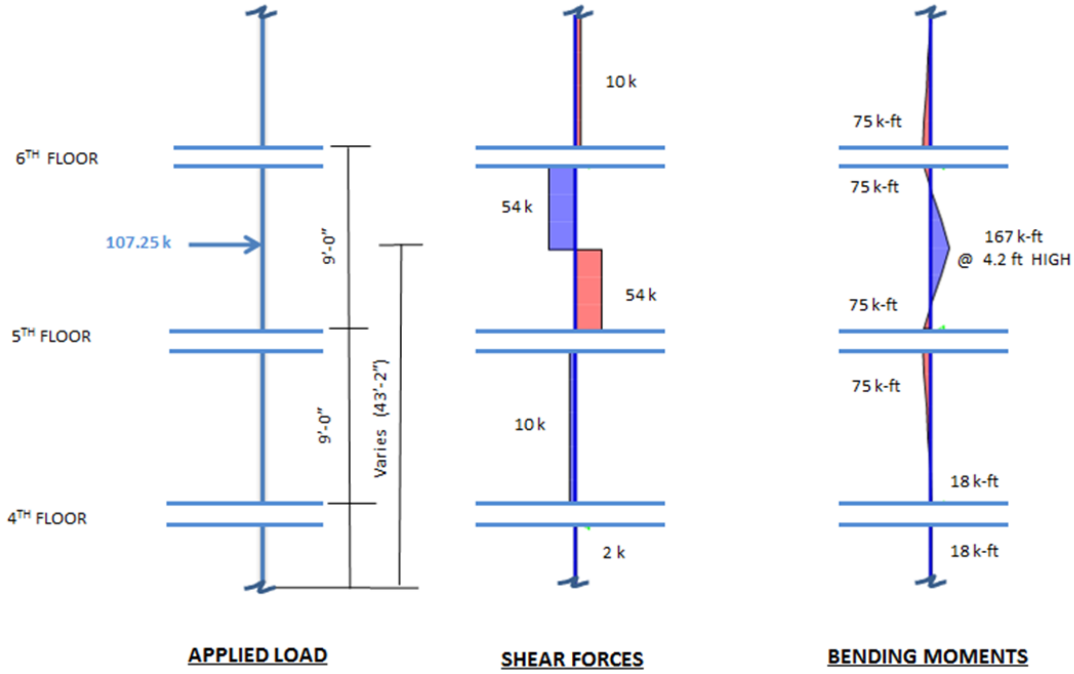


Figure C-101: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 5th floor column

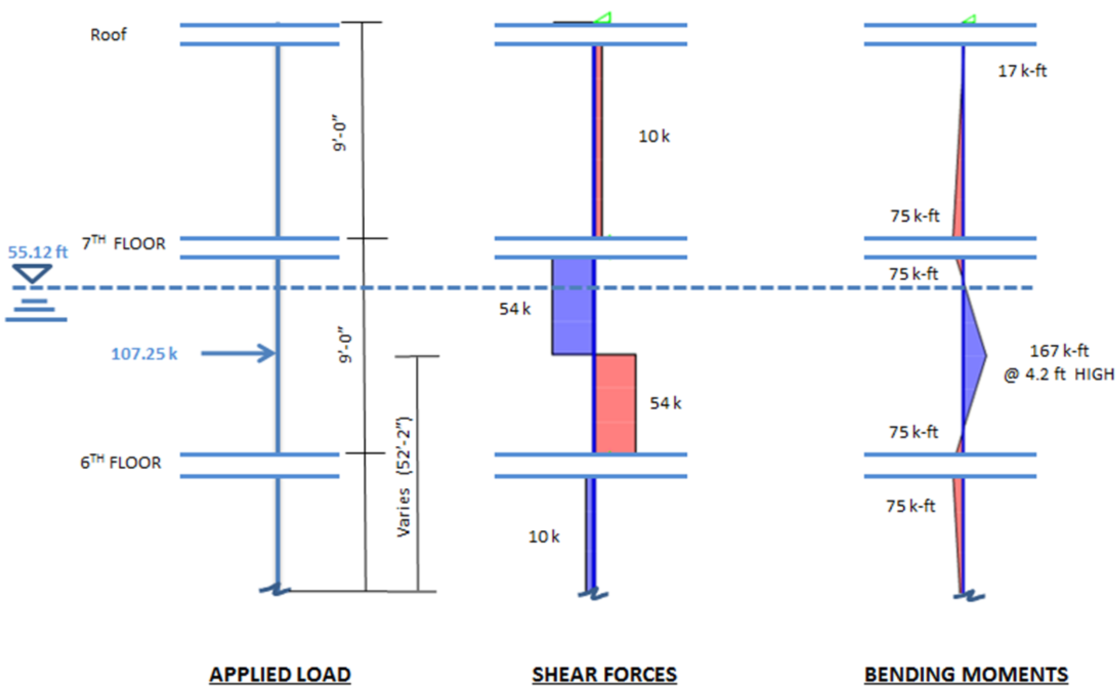


Figure C-102: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 6th floor column

Table C-6 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro)

and long impact (Impact). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table C-6: Results from loading conditions of Hilo residential building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
1170	514.3	440	282	1.2D+Ftsu+0.5L (Hydro)
1170	365.4	440	282	0.9D+Ftsu (Hydro)
191	514.3	95	76	1.2D+Ftsu+0.5L (Impact)
191	365.4	95	76	0.9D+Ftsu (Impact)
Floor 2				
872	440.8	290	132	1.2D+Ftsu+0.5L (Hydro)
872	313.2	290	132	0.9D+Ftsu (Hydro)
168	440.8	92	70	1.2D+Ftsu+0.5L (Impact)
168	313.2	92	70	0.9D+Ftsu (Impact)
Floor 3				
700	367.4	271	113	1.2D+Ftsu+0.5L (Hydro)
700	261	271	113	0.9D+Ftsu (Hydro)
167	367.4	92	69	1.2D+Ftsu+0.5L (Impact)
167	261	92	69	0.9D+Ftsu (Impact)
Floor 4				
700	293.9	287	129	1.2D+Ftsu+0.5L (Hydro)
700	208.8	287	129	0.9D+Ftsu (Hydro)
167	293.9	92	69	1.2D+Ftsu+0.5L (Impact)
167	208.8	92	69	0.9D+Ftsu (Impact)
Floor 5				
252	220.4	35	35	1.2D+Ftsu+0.5L (Hydro)
252	156.6	35	35	0.9D+Ftsu (Hydro)
167	220.4	92	69	1.2D+Ftsu+0.5L (Impact)
167	156.6	92	69	0.9D+Ftsu (Impact)
Floor 6				
60	146.9	8	8	1.2D+Ftsu+0.5L (Hydro)
60	104.4	8	8	0.9D+Ftsu (Hydro)
167	146.9	92	69	1.2D+Ftsu+0.5L (Impact)
167	104.4	92	69	0.9D+Ftsu (Impact)
Floor 7				
14	73.5	2	2	1.2D+Ftsu+0.5L (Hydro)
14	52.2	2	2	0.9D+Ftsu (Hydro)
75	73.5	10	10	1.2D+Ftsu+0.5L (Impact)
75	52.2	10	10	0.9D+Ftsu (Impact)

C.15.1.2 Existing Exterior Column Design for Combined Gravity and Tsunami Loads

The column chosen at Grid Intersection A-3 from **Figure C-17** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure C-103 to **Figure C-107** shows the interaction diagram for the typical exterior column including the tsunami load combinations.

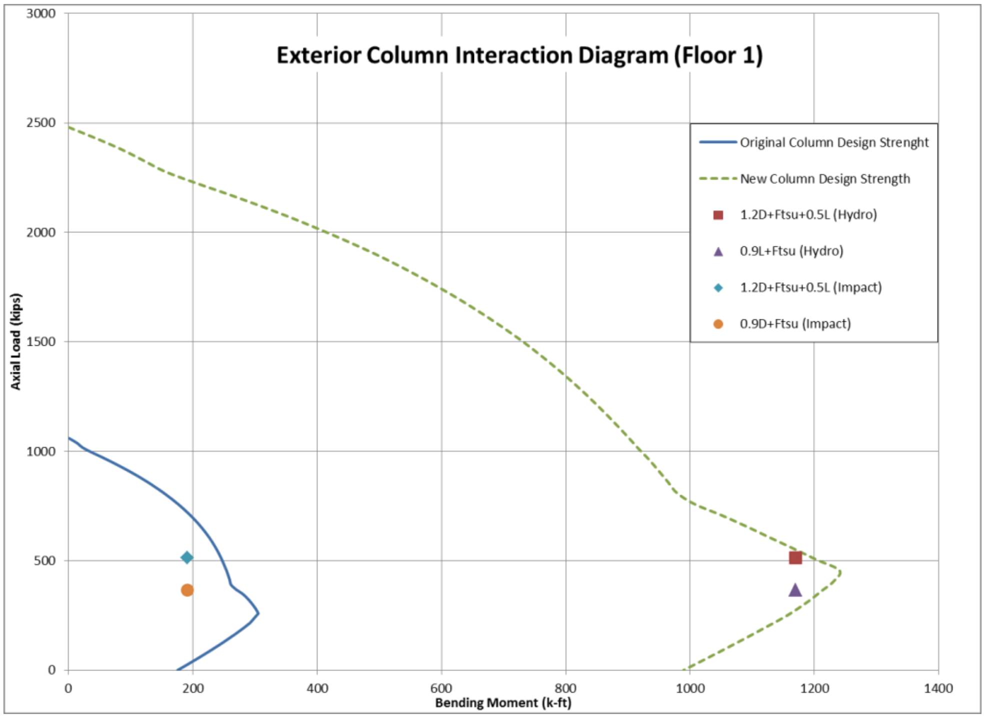


Figure C-103: Interaction diagram for typical ground floor exterior column showing tsunami load combinations

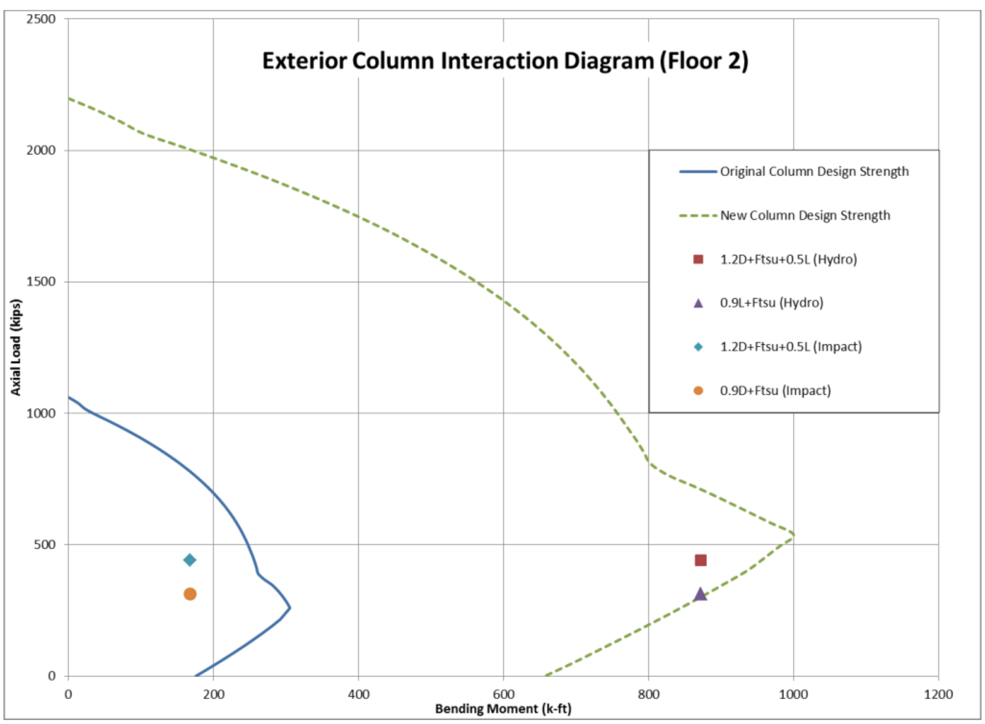


Figure C-104: Interaction diagram for typical 2nd floor exterior column showing tsunami load combinations

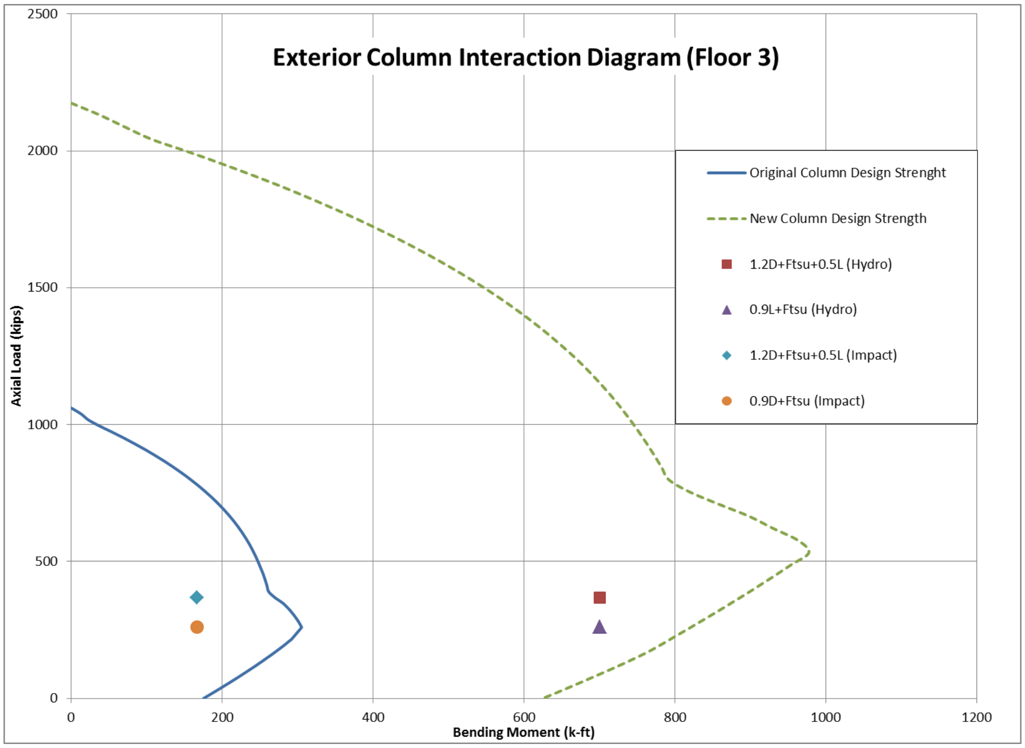


Figure C-105: Interaction diagram for typical 3rd floor exterior column showing tsunami load combinations

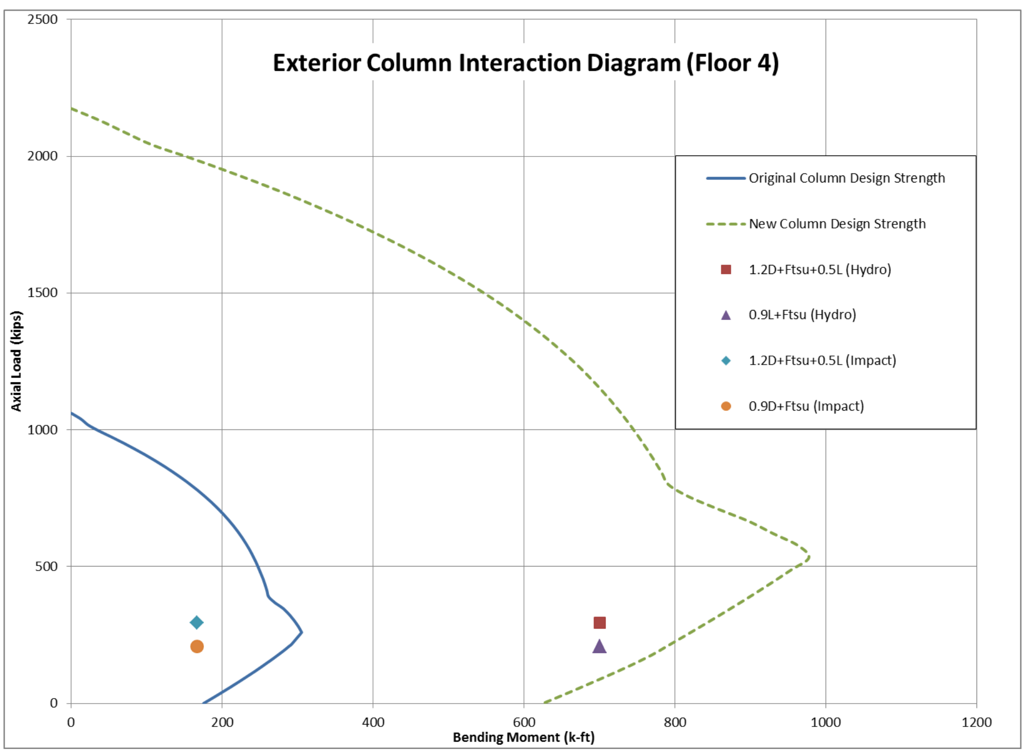


Figure C-106: Interaction diagram for typical 4th floor exterior column showing tsunami load combinations

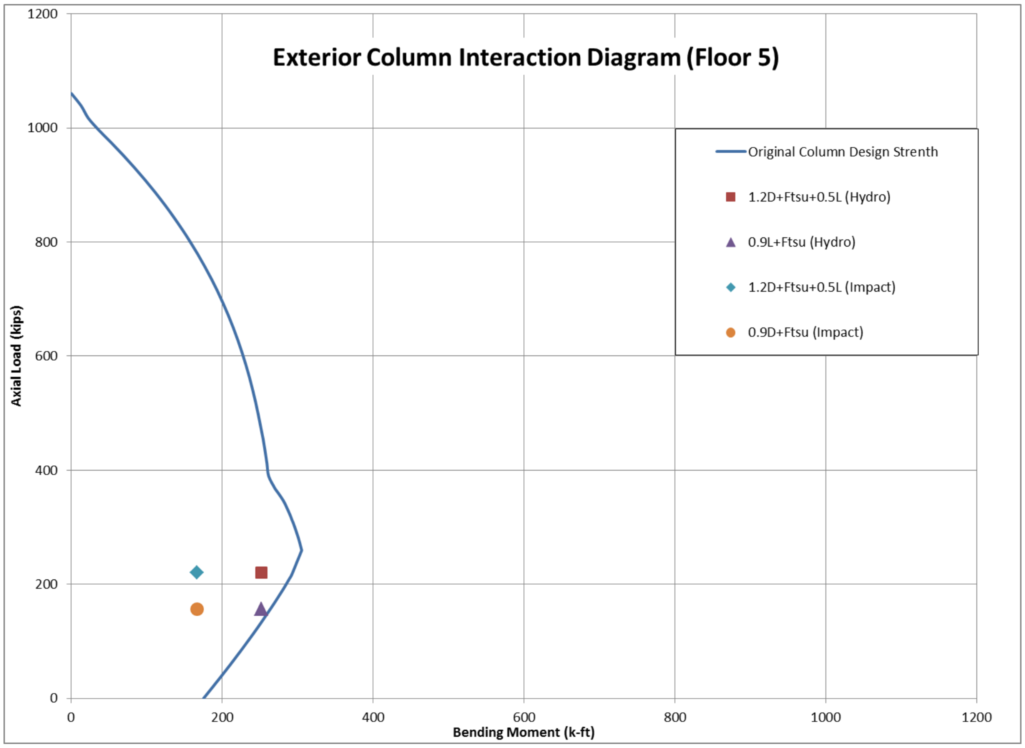


Figure C-107: Interaction diagram for typical 5th floor exterior column showing tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

C.15.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure C-103** to **Figure C-107** the original exterior columns are adequate for log impact load, but the columns at the ground and 2nd floors must be strengthened to resist bending due to the hydrodynamic loads. Revised columns designs were developed to satisfy the hydrodynamic loads as shown in in **Figure C-108** to **Figure C-115**. The interaction diagrams for these new columns are shown in **Figure C-103** to **Figure C-106**.

Floor 1

End Section (A)

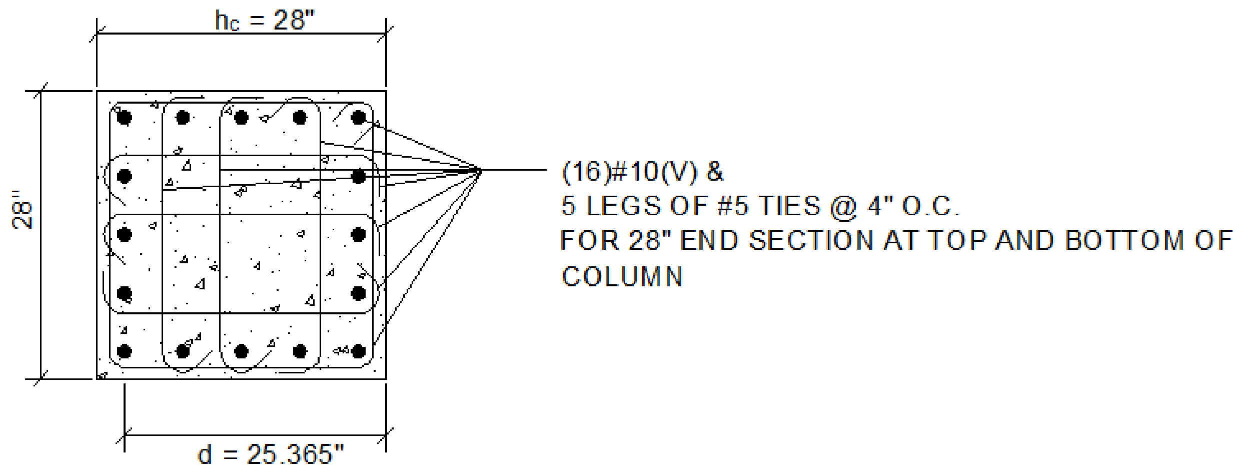


Figure C-108: Exterior column, cross-section at end of column at ground floor level based on tsunami design requirements.

Center Section (B)

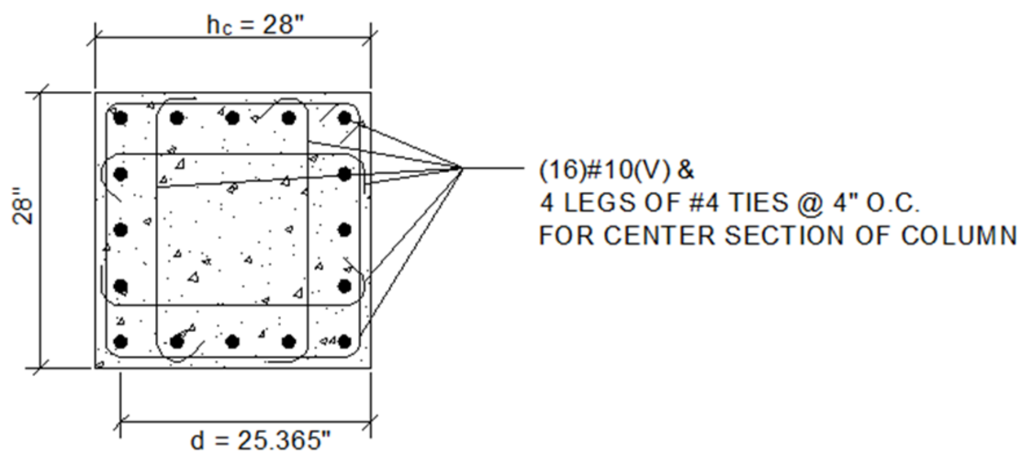


Figure C-109: Exterior column, cross-section at center of column at ground floor level based on tsunami design requirements.

Floor 2

End Section (A)

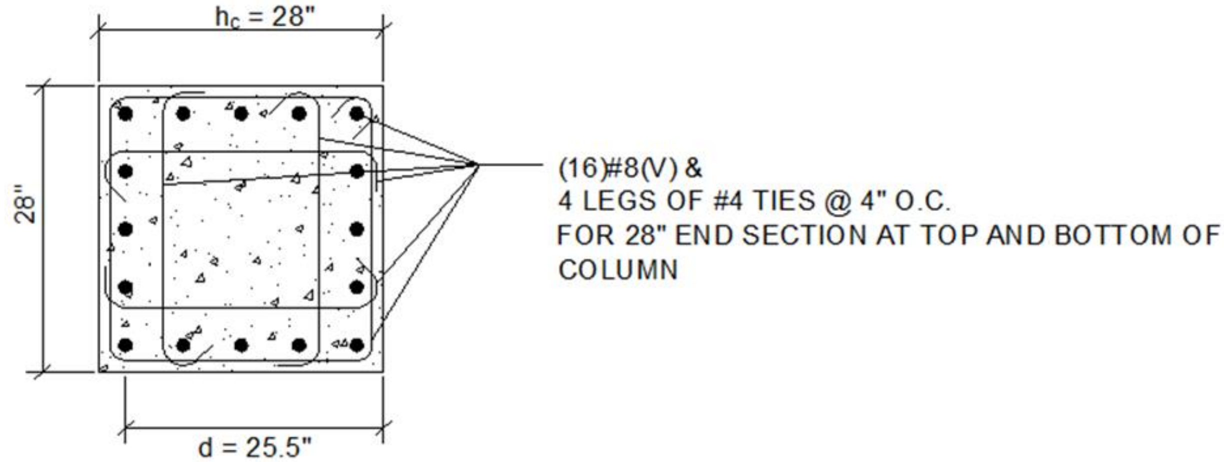


Figure C-110: Exterior column, cross section at end of column at the 2nd floor level based on tsunami design requirements.

Center Section (B)

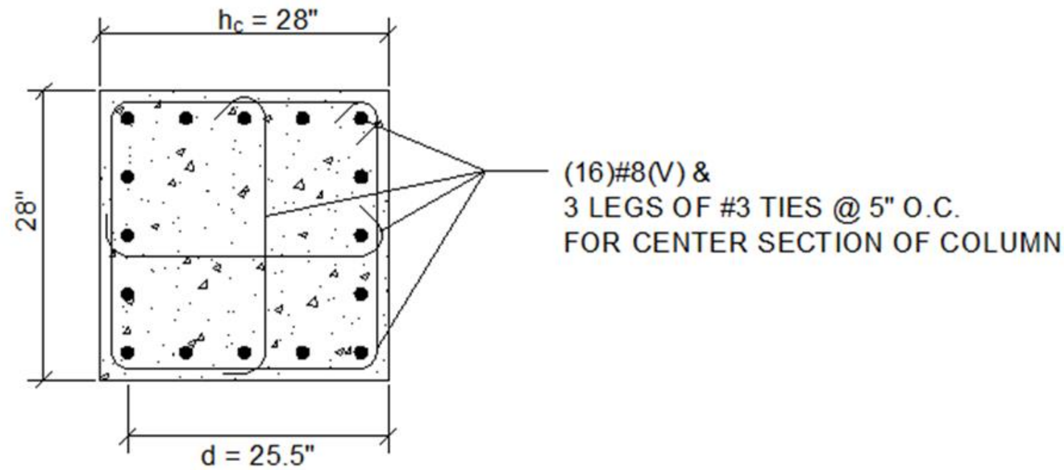


Figure C-111: Exterior column, cross-section at center of column at the 2nd floor level based on tsunami design requirements.

Floor 3

End Section (A)

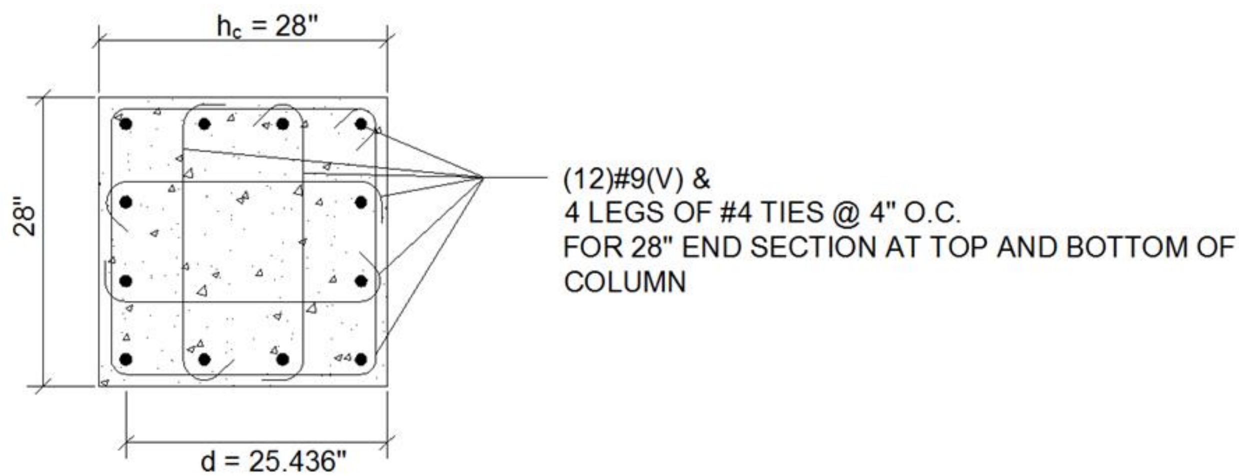


Figure C-112: Exterior column, cross section at end of column at the 3rd floor level based on tsunami design requirements.

Center Section (B)

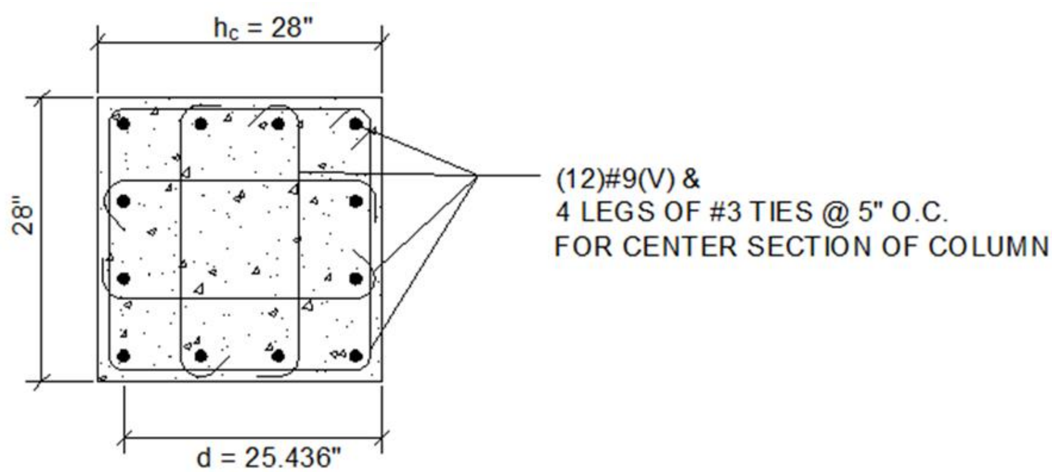


Figure C-113: Exterior column, cross-section at center of column at the 3rd floor level based on tsunami design requirements.

Floor 4

End Section (A)

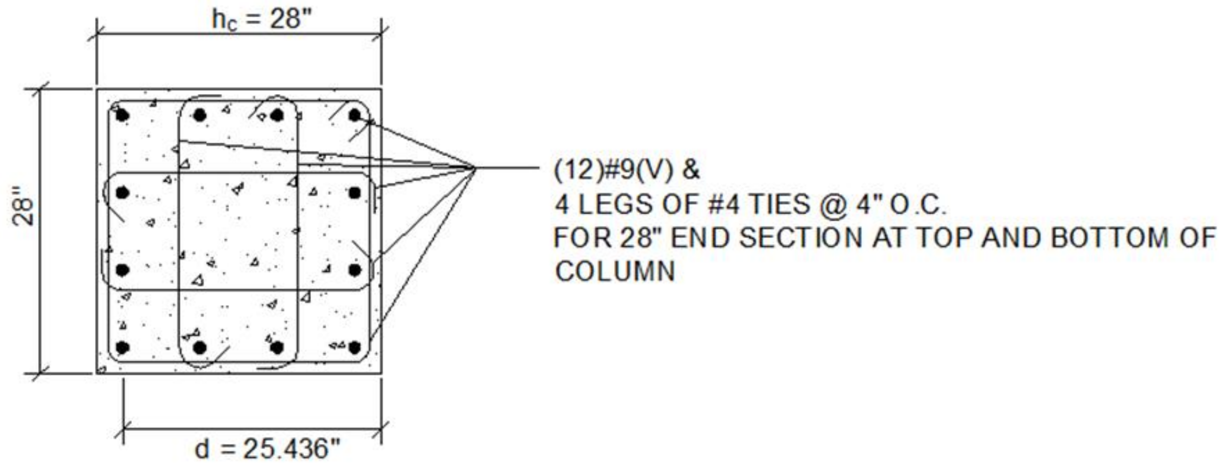


Figure C-114: Exterior column, cross section at end of column at the 4th floor level based on tsunami design requirements.

Center Section (B)

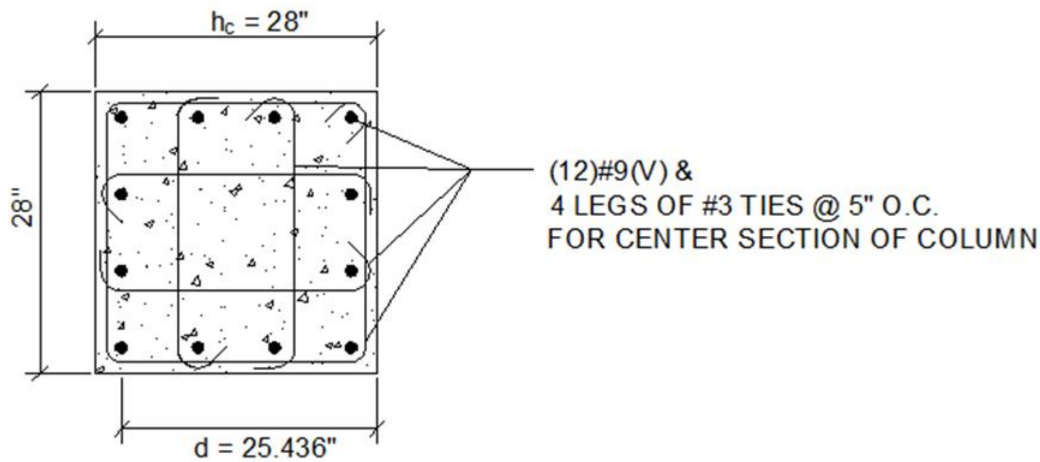


Figure C-115: Exterior column, cross-section at center of column at the 4th floor level based on tsunami design requirements.

C.15.1.4 Exterior Column Shear Design

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = 365.4 kips.

The shear capacities of the 28"x28" columns with 4 leg #4 Stirrups at 4" o.c. in the end section and 5 leg #5 Stirrups at 4" o.c. in the center section are given by:

$$\text{Ø}V_n = \text{Ø}(V_C + V_S)$$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{6{,}500}\left(1 + \frac{365{,}400}{2{,}000\times28\times28}\right)28 \times 25.365/1{,}000 = 141$ kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(5\times0.31)\times60{,}000\times25.365}{4\times1{,}000} = 590$ kips

$$V_S = \frac{A_v f_y d}{s} = 590 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 28 \times 25.365 = 458 \text{ kips} \therefore \text{use } 458 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times25.365}{5\times1{,}000} = 304$ kips.

$$V_S = \frac{A_v f_y d}{s} = 304 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 28 \times 25.365 = 458 \text{ kips} \therefore \text{use } 304 \text{ kips}$$

Therefore in the end sections, $ØV_n = 0.75\ (141 + 458) = 450$ k

and in the center sections, $ØV_n = 0.75\ (141 + 304) = 334$ k.

At d: $V_u = 440$ k $< \phi V_n = 450$ k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: $V_u = 282$ k $< \phi V_n = 334$ k, therefore the column is adequate for shear at the center section.

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of $(0.9D + F_{TSU})$ per **Eqn. 6.8.3.3.-1b**, $P_u = 313.2$ kips.

The shear capacities of the 28”x28” columns with 4 leg #4 Stirrups at 4” o.c. in the end section and 3 leg #3 Stirrups at 5” o.c. in the center section are given by:

$$ØV_n = Ø(V_C + V_S)$$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{208{,}800}{2{,}000\times28\times28}\right)28 \times 25.5/1{,}000 = 108$ kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times25.5}{4\times1{,}000} = 306$ kips

$$V_S = \frac{A_v f_y d}{s} = 306 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 28 \times 25.5 = 361 \text{ kips} \therefore \text{use } 306 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times25.5}{5\times1{,}000} = 101$ kips.

$$V_S = \frac{A_v f_y d}{s} = 101 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 28 \times 25.5 = 361 \text{ kips} \therefore \text{use } 101 \text{ kips}$$

Therefore in the end sections, $ØV_n = 0.75\ (108 + 306) = 311$ k

and in the center sections, $ØV_n = 0.75\ (108 + 101) = 157$ k.

At d: $V_u = 290$ k $< \phi V_n = 311$ k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: V_u = 132 k < ϕV_n = 157 k, therefore the column is adequate for shear at the center section.

Critical Shears in Columns at 3rd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 261 kips.

The shear capacities of the 28”x28” columns with 4 leg #4 Stirrups at 4” o.c. in the end section and 4 leg #3 Stirrups at 5” o.c. in the center section are given by:

$Ø V_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{261{,}000}{2{,}000\times28\times28}\right)28\times25.436/1{,}000$ = 105 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times25.436}{4\times1{,}000}$ = 305 kips

$V_S = \frac{A_v f_y d}{s}$ = 305 kips $\ngtr 8\sqrt{f_C'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}000}\times28\times25.436 = 360$ kips $\therefore$ use 305 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.11)\times60{,}000\times25.436}{5\times1{,}000}$ = 134 kips.

$V_S = \frac{A_v f_y d}{s}$ = 134 kips $\ngtr 8\sqrt{f_C'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}000}\times28\times25.436 = 360$ kips $\therefore$ use 134 kips

Therefore in the end sections, $ØV_n$ = 0.75 (105 + 305) = 308 k

and in the center sections, $ØV_n$ = 0.75 (105 + 134) = 180 k.

At d: V_u = 271 k < ϕV_n = 308 k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: V_u = 113 k < ϕV_n = 180 k, therefore the column is adequate for shear at the center section.

Critical Shears in Columns at 4th Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 208.8 kips.

The shear capacities of the 28”x28” columns with 4 leg #4 Stirrups at 4” o.c. in the end section and 3 leg #3 Stirrups at 5” o.c. in the center section are given by:

$Ø V_n = Ø(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{208{,}800}{2{,}000\times28\times28}\right)28\times25.436/1{,}000$ = 102 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times25.436}{4\times1{,}000}$ = 305 kips

$V_S = \frac{A_v f_y d}{s}$ = 305 kips $\ngtr 8\sqrt{f_C{}'}$ b d $= 8 \times \sqrt{4{,}000} \times 28 \times 25.436 = 360$ kips $\therefore$ use 305 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.11)\times60{,}000\times25.436}{5\times1{,}000}$ = 134 kips.

$V_S = \frac{A_v f_y d}{s}$ = 134 kips $\ngtr 8\sqrt{f_C{}'}$ b d $= 8 \times \sqrt{4{,}000} \times 28 \times 25.436 = 360$ kips $\therefore$ use 134 kips

Therefore in the end sections, ϕV_n = 0.75 (102 + 305) = 305 k

and in the center sections, ϕV_n = 0.75 (102 + 134) = 177 k

At d: V_u = 287 k < ϕV_n = 305 k, therefore the column is adequate for shear at the end sections.

At d + h_c: V_u = 129 k < ϕV_n = 177 k, therefore the column is adequate for shear at the center section.

Critical Shears in Columns at 5th Floor:

At the critical axial load combination of (0.9*D* + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 156.6 kips.

The shear capacities of the 20”x20” columns with 3 leg #4 Stirrups at 4” o.c. in the end section and 3 leg #3 Stirrups at 5” o.c. in the center section are given by:

$\phi V_n = \phi(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{156{,}600}{2{,}000\times20\times20}\right)20 \times 17.5625/1{,}000$ = 53 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times17.5625}{4\times1{,}000}$ = 158 kips

$V_S = \frac{A_v f_y d}{s}$ = 158 kips $\ngtr 8\sqrt{f_C{}'}$ b d $= 8 \times \sqrt{4{,}000} \times 20 \times 17.5625 = 178$ kips $\therefore$ use 158 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times17.5625}{5\times1{,}000}$ = 70 kips.

$V_S = \frac{A_v f_y d}{s}$ = 70 kips $\ngtr 8\sqrt{f_C{}'}$ b d $= 8 \times \sqrt{4{,}000} \times 20 \times 17.5625 = 178$ kips $\therefore$ use 70 kips

Therefore in the end sections, ϕV_n = 0.75 (53 + 158) = 158 k

and in the center sections, ϕV_n = 0.75 (53 + 70) = 92 k

At d: V_u = 92 k ≤ ϕV_n = 92 k, therefore the column is adequate for shear at the end sections.

At d + h_c: V_u = 69 k < ϕV_n = 158 k, therefore the column is adequate for shear at the center section.

By inspection the remaining columns are adequate to resist the tsunami shear force.

C.15.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure C-17**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure A-82** and **Figure A-83** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the ends of the column, where *d* = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "$d + h_c$" from each end of the column, where $d + h_c = 17.5 + 20 = 37.5$ in.

Floor 1 – 7

End Section (A)

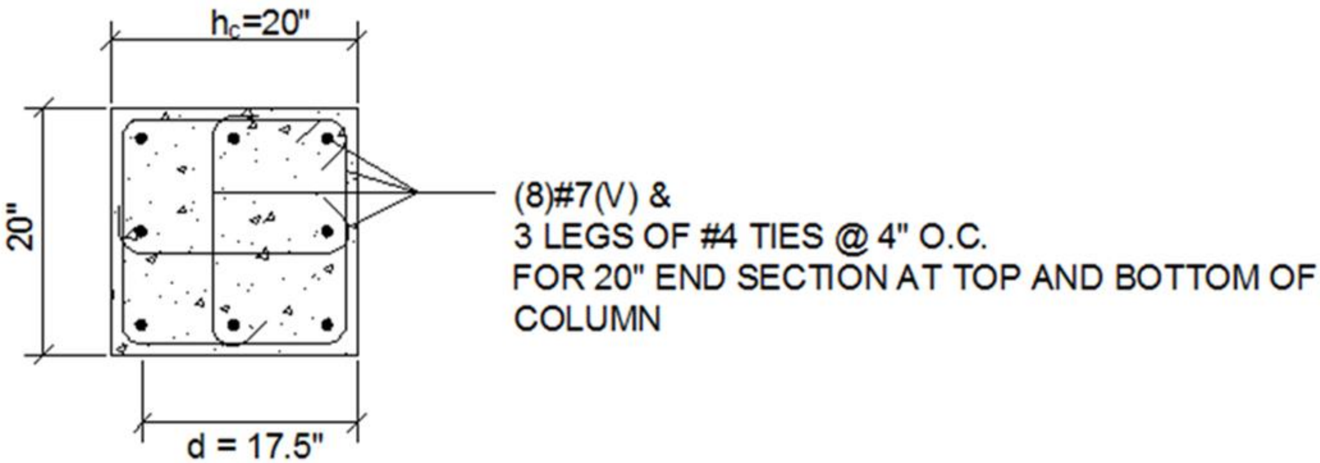


Figure C-116: Interior column, cross-section end of column at all floor levels based on SDC D design.

Center Section (B)

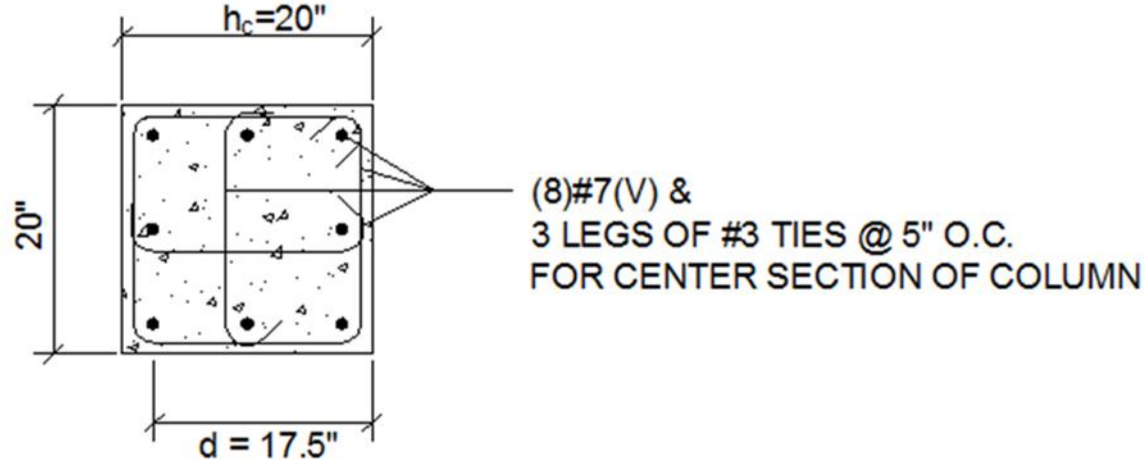


Figure C-117: Interior column, cross-section at center of column at all floor levels based on SDC D design.

C.15.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 17.83 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$$P_D = [128(14.58)(28)(6)+ 123(17.83)(28)+1.67^2(150)(66)]/1000 = 472\ k$$

Floor Live load reduction factor = $0.25+15/[4(17.83)(28)(6)]^{0.5} = 0.487$, therefore, column base live load is:

$$P_L = 0.487[55(17.83)(28)(6)]/1000 = 80.2\ k$$

Roof Live Load reduction factor = $R_1R_2= [1.2-(0.001)(17.83)(28)](1.0) = 0.701$, column roof live load is:

$$P_{Lr} = 0.701(20)(17.83)(28)/1000 = 6.61\ k$$

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

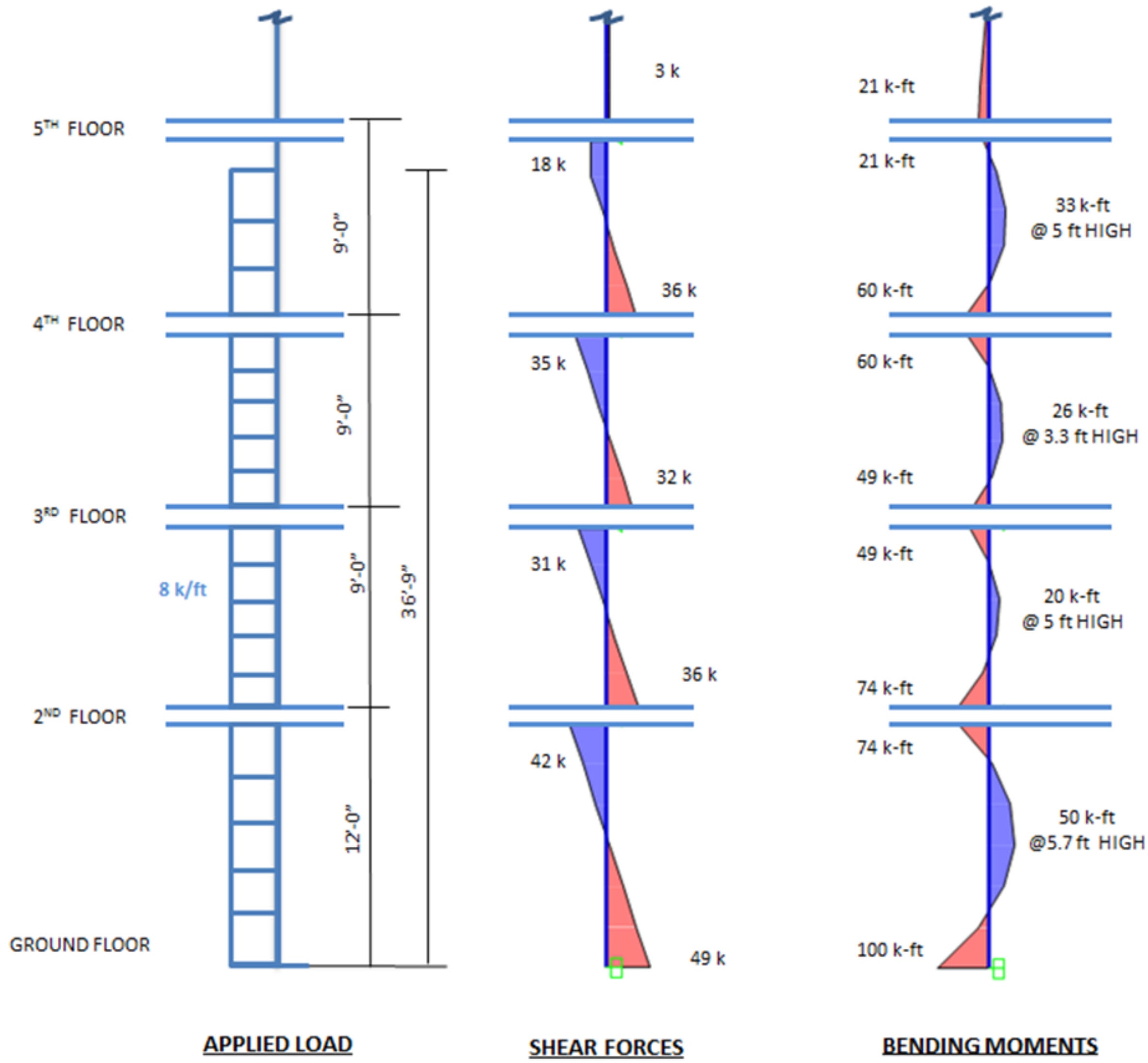


Figure C-118: Hydrodynamic loading on interior column of Hilo residential building due to Load Case 2

Table C-7 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table C-7: Results from loading conditions of Hilo residential building interior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
100	811.8	37	24	1.2D+Ftsu+0.5L (Hydro)
100	566.1	37	24	0.9D+Ftsu (Hydro)
Floor 2				
74	676.5	25	11	1.2D+Ftsu+0.5L (Hydro)
74	471.8	25	11	0.9D+Ftsu (Hydro)
Floor 3				
60	541.2	23	10	1.2D+Ftsu+0.5L (Hydro)
60	377.4	23	10	0.9D+Ftsu (Hydro)
Floor 4				
60	405.9	24	11	1.2D+Ftsu+0.5L (Hydro)
60	283.1	24	11	0.9D+Ftsu (Hydro)
Floor 5				
21	270.6	3	3	1.2D+Ftsu+0.5L (Hydro)
21	188.7	3	3	0.9D+Ftsu (Hydro)
Floor 6				
5	135.3	1	1	1.2D+Ftsu+0.5L (Hydro)
5	94.4	1	1	0.9D+Ftsu (Hydro)
Floor 7				
1	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
1	94.4	0	0	0.9D+Ftsu (Hydro)

C.15.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure C-17** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure C-119 shows the interaction diagram for the typical exterior column including the tsunami load combinations.

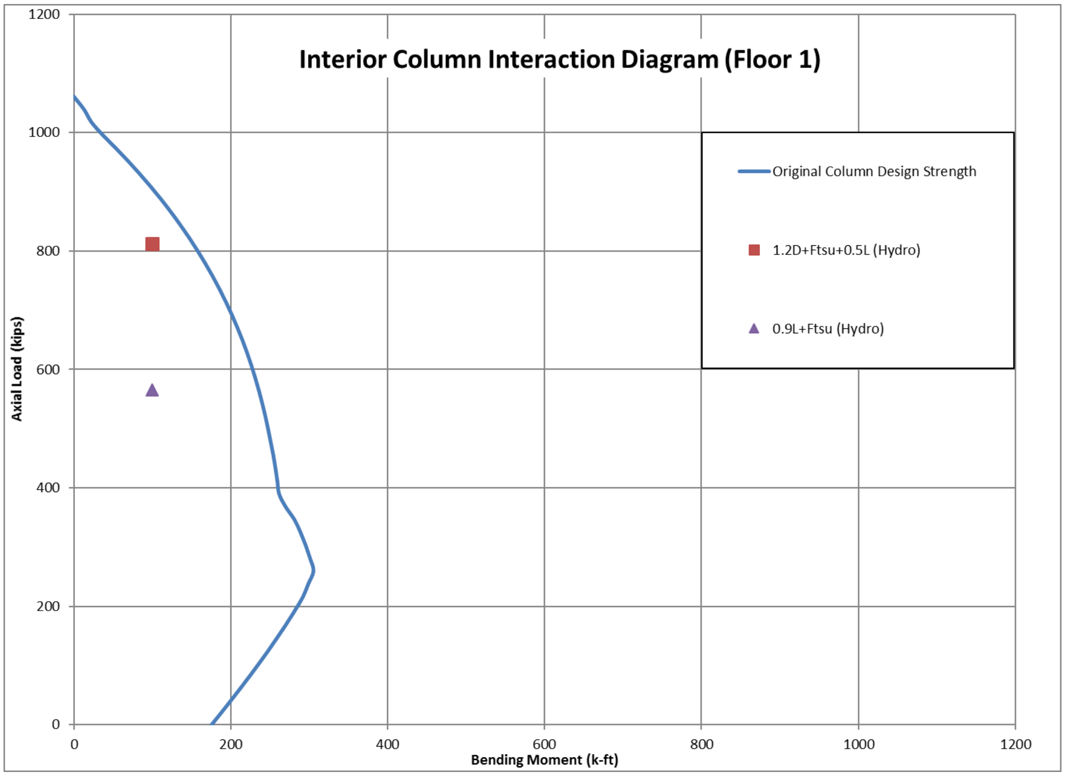


Figure C-119: Interaction diagram for typical ground floor residential interior column showing tsunami load combinations

C.15.2.3 Interior Column Shear Design

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of ($1.2D + F_{TSU} + 0.5L$) per **Eqn. 6.8.3.3.-1a,** P_u = 811.8 kips.

The shear capacities of the existing 20”x20” column with 3 leg #4 Stirrups @ 4” o.c. in the end sections and 3 leg #3 Stirrups @ 5” o.c. in the center section are given by:

$$\text{Ø}V_n = \text{Ø}(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{811{,}800}{2{,}000\times 20\times 20}\right)20 \times 17.5625/1{,}000 = 90$ kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.2)\times 60{,}000\times 17.5625}{4\times 1{,}000} = 158$ kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.11)\times 60{,}000\times 17.5625}{5\times 1{,}000} = 70$ kips

Therefore in the end sections, $\text{Ø}V_n = 0.75\,(90 + 158) = 186$ k

and in the center sections, $\text{Ø}V_n = 0.75\,(90 + 70) = 120$ k

At d: V_u = 37 k < ϕV_n = 186 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 24 k < ϕV_n = 120 k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear force.

C.15.3 Typical Exterior Wall Design

A section of exterior wall along Grid Line D from **Figure C-17** adjacent to the mechanical room was analyzed. The wall is part of the lateral resisting system for seismic loads, acting as a shear wall for longitudinal forces and boundary element for transverse forces. Seismic Design Category D design and detailing of the 10" thick wall resulted in the reinforcement layout shown in **Figure C-120** to **Figure A-88**. The wall will now be checked for tsunami loads.

For comparative purposes with the debris impact loads, the ultimate shear forces and bending moments are provided for an effective width of wall equal to 5.67 ft. The critical shear force occurs at a distance "*d*" from the base of the wall, where *d* = 10-0.75-1"/2 = 8.75 in.

Floor 1

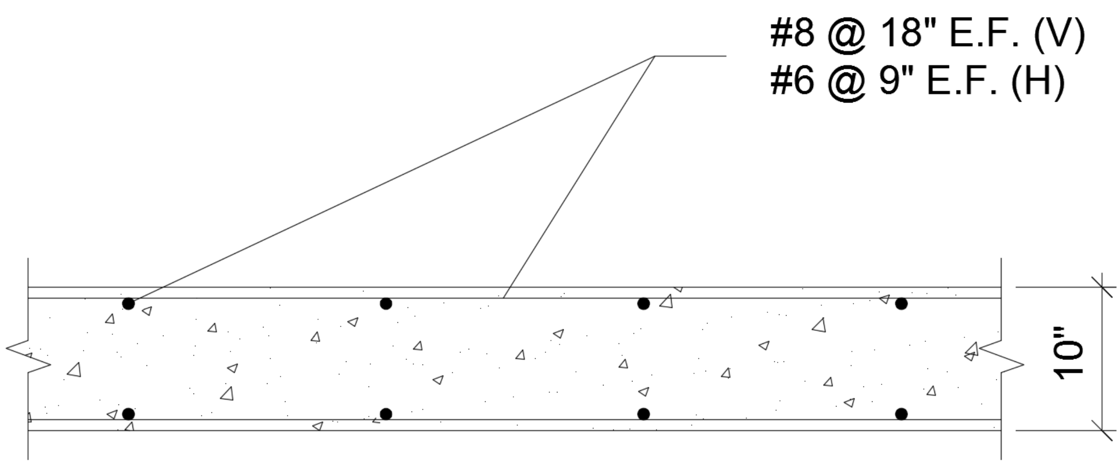


Figure C-120: Segment of exterior wall cross-section at the ground floor level based on SDC D design.

Floor 2

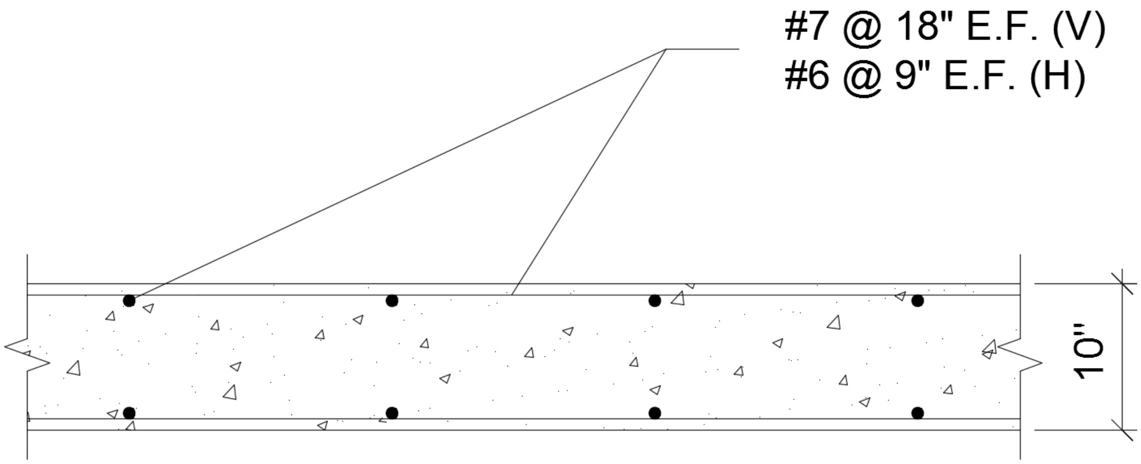


Figure C-121: Segment of exterior wall cross-section at the 2nd floor level based on SDC D design.

Floor 3 – 7

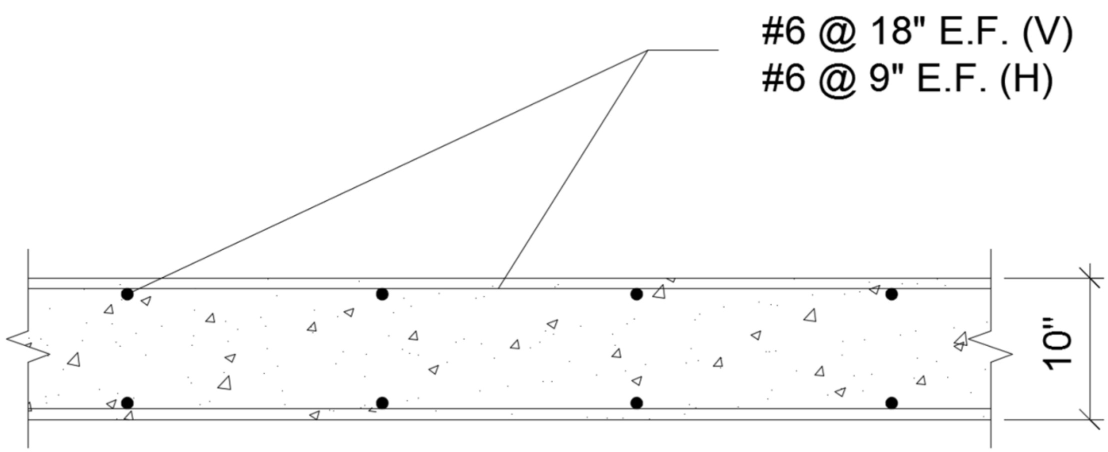


Figure C-122: Segment of exterior wall cross-section at the 3rd – 7th floor level based on SDC D design.

C.15.3.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 5.5 ft. Gravity loads will be computed per foot width of wall.

The Dead Load at the base of the wall is:

$P_D = [128(5.5)(1)(6)+123(5.5)(1)+0.83(1)(150)(66)]/1000 = 13.1$ k/ft

Floor Live load reduction: Reduction Factor = $0.25+15/[1(5.5)(28)(6)]^{0.5} = 0.743$

$P_L = 0.743[55(5.5)(1)(6)]/1000 = 1.35$ k/ft

Roof Live Load reduction: Reduction Factor = $R_1R_2= [1.2-(0.001)(5.5)(28)](1.0) = 1.05$*, Use 1.0*

$P_{Lr} = 20(5.5)(1)/1000 = 0.110$ k/ft

Analysis of a 5.67 foot width of wall with the applied tsunami hydrodynamic loads from Load Case 2 results in the following bending moment and shear force diagrams:

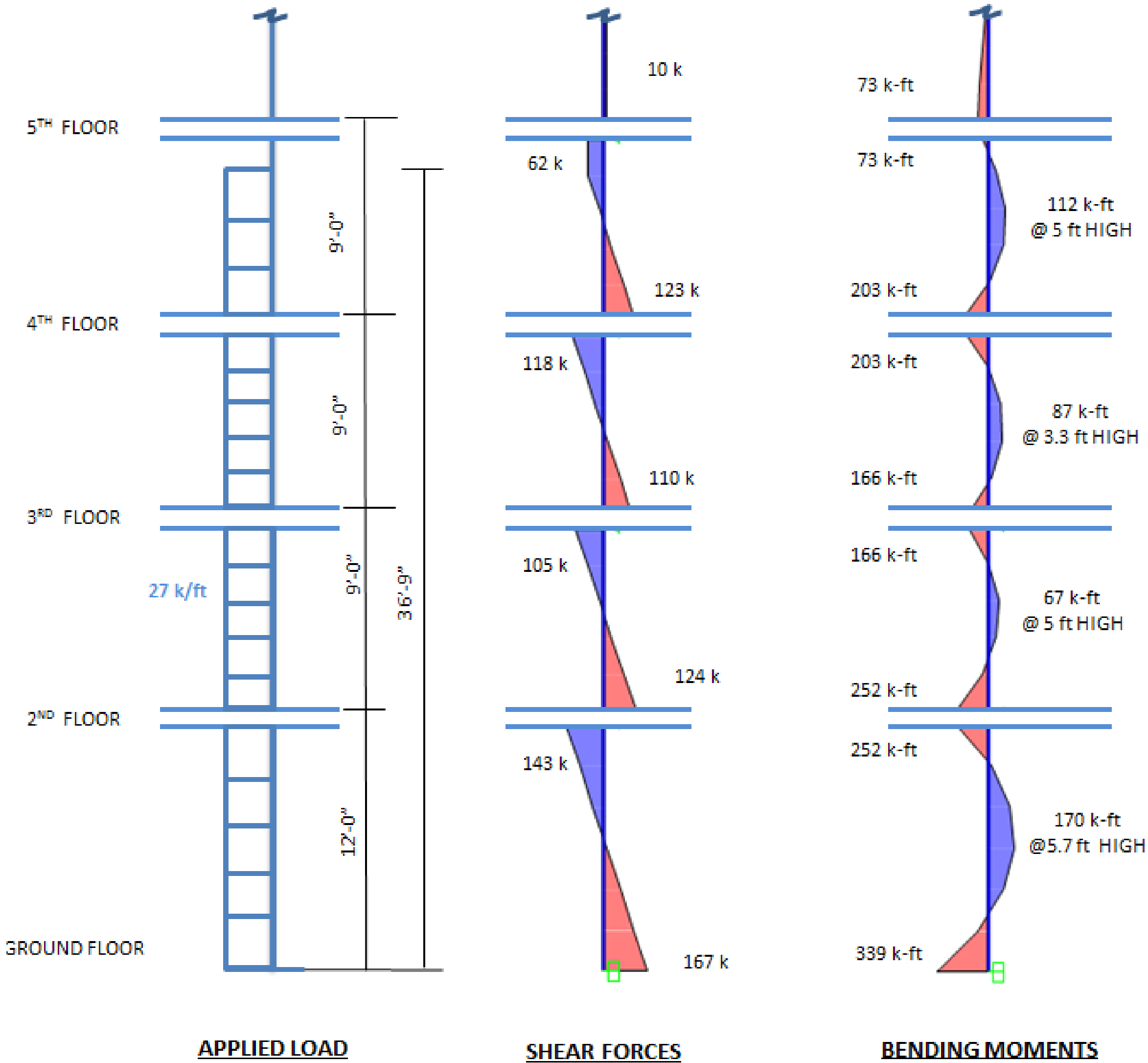


Figure C-123: Hydrodynamic loading on exterior wall of Hilo residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike, acts over an effective width of 5.67 ft, at a point just below the slab at each inundated floor for maximum shear and at the mid-height of the clear column height for maximum bending moments. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure C-124** to **Figure C-129**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown in **Figure C-130** to **Figure C-135**.

Impact load at d:

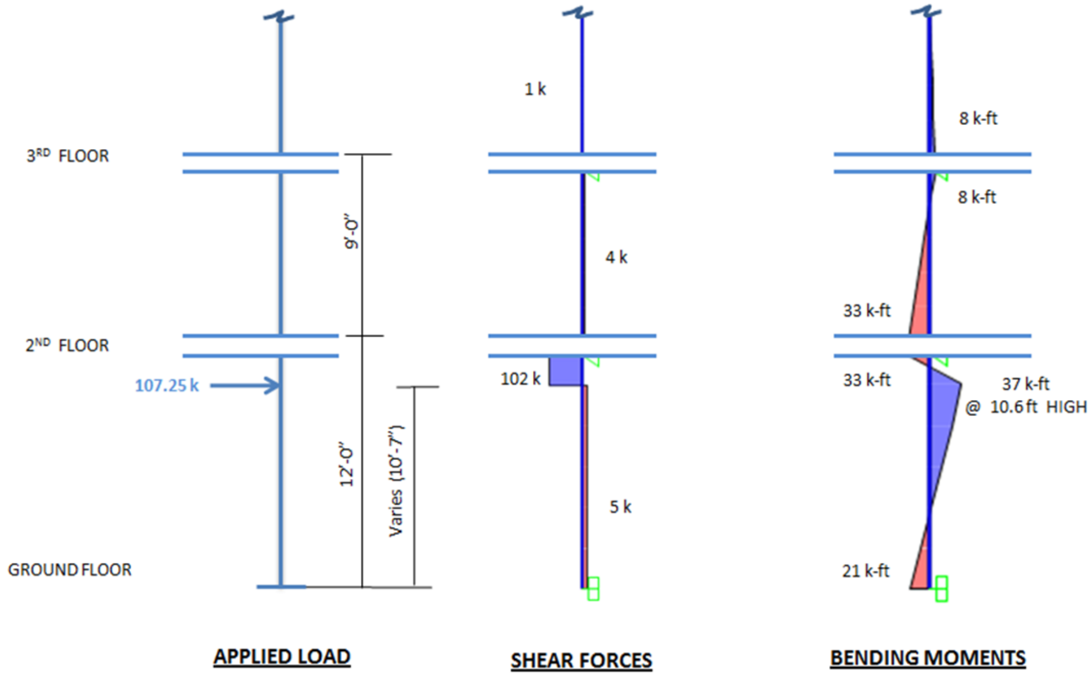


Figure C-124: Impact load applied at d away from the end of column on the ground floor

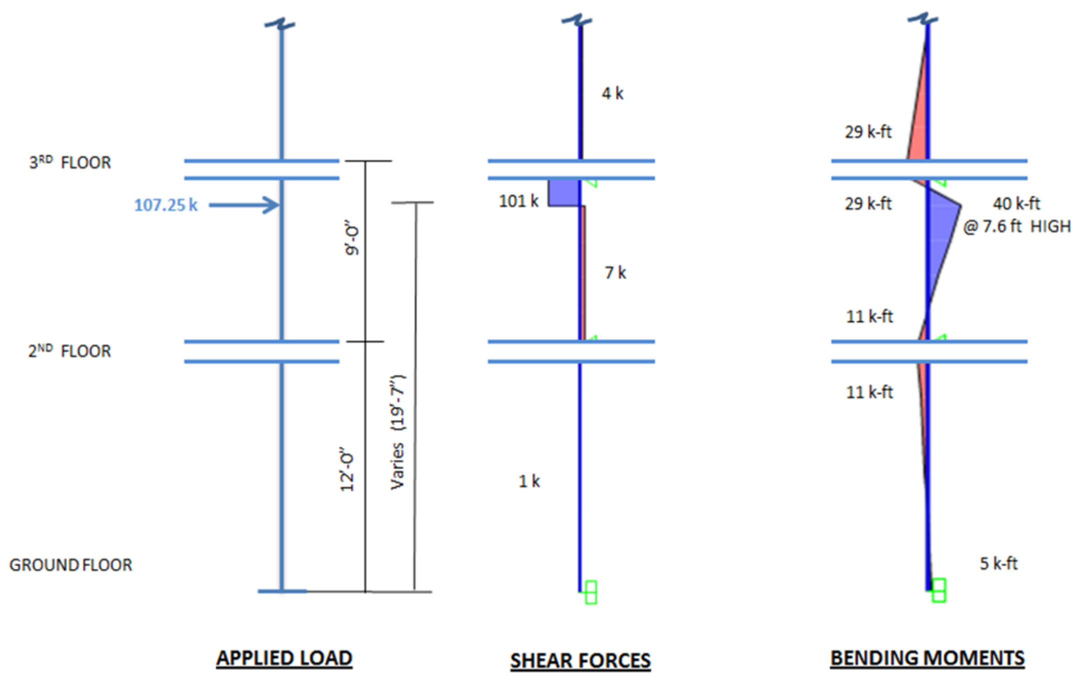


Figure C-125: Impact load applied at d away from the end of column on the 2nd floor

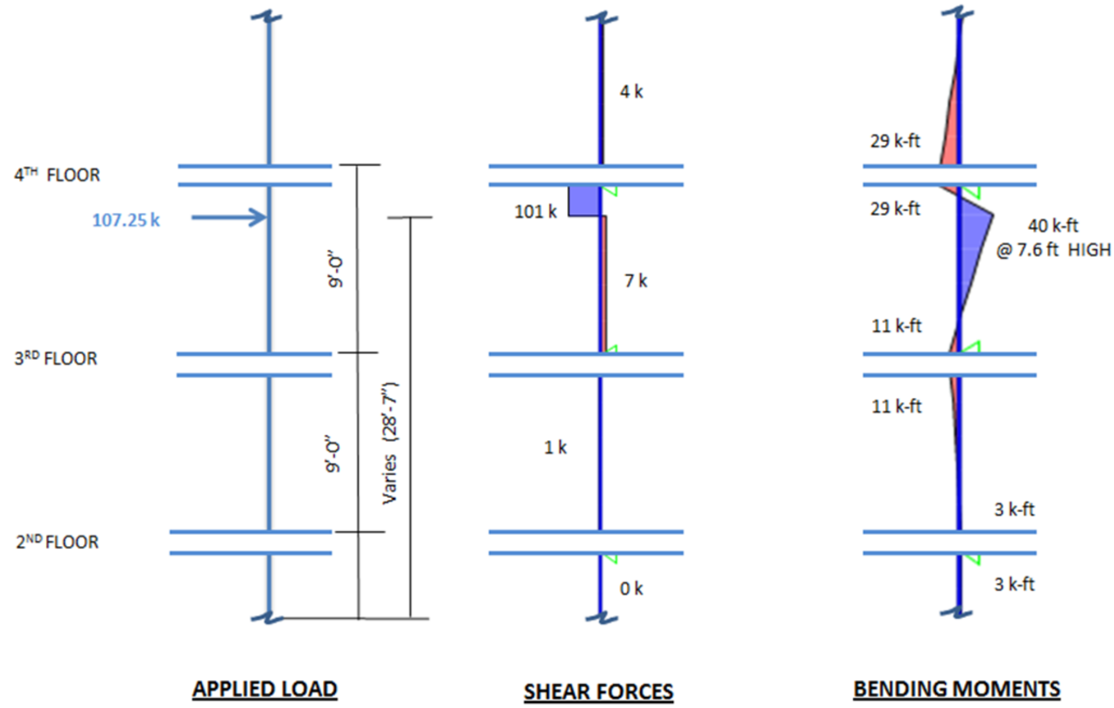


Figure C-126: Impact load applied at d away from the end of column on the 3rd floor

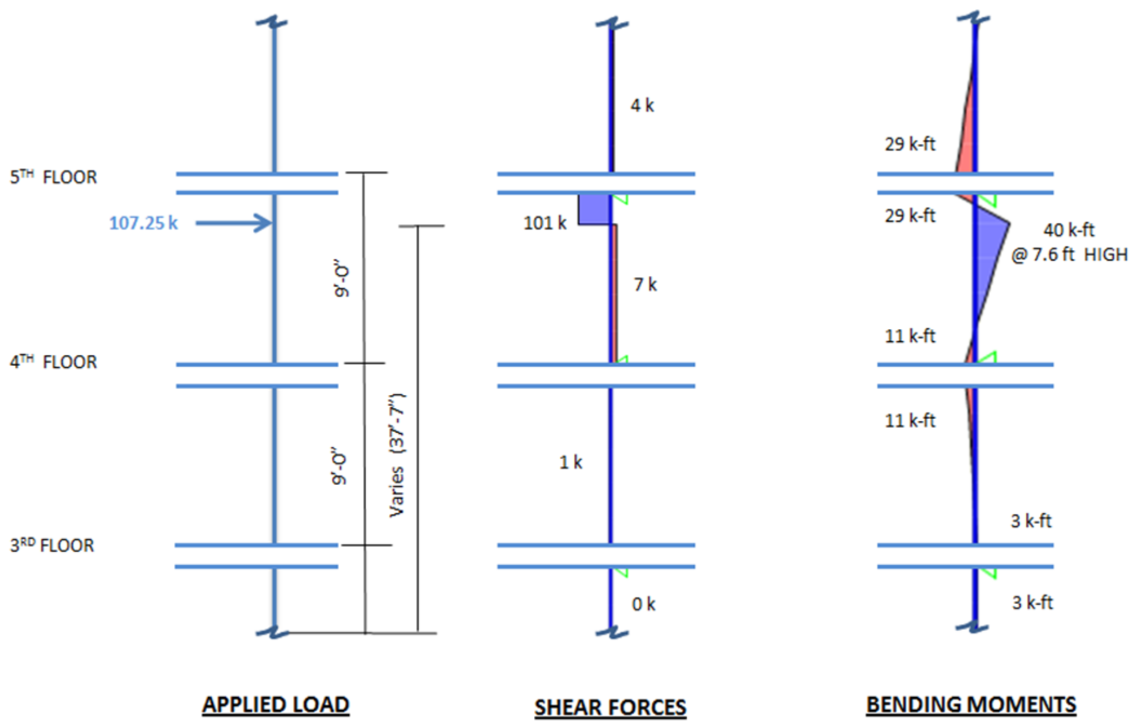


Figure C-127: Impact load applied at d away from the end of column on the 4th floor

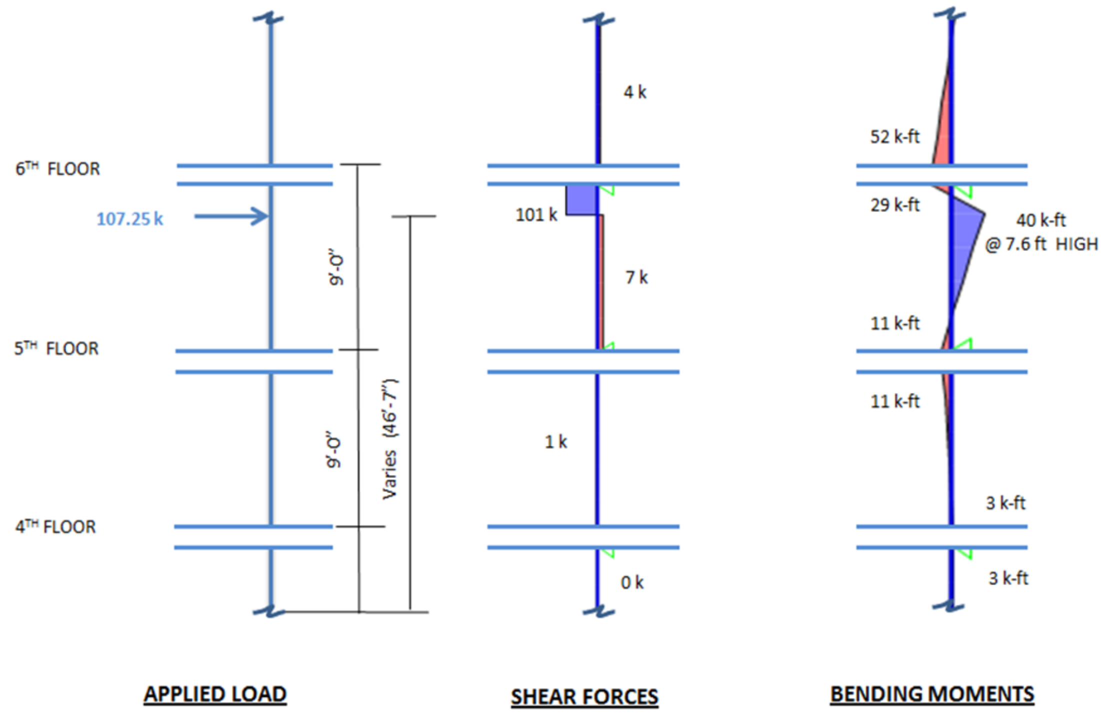


Figure C-128: Impact load applied at d away from the end of column on the 5th floor

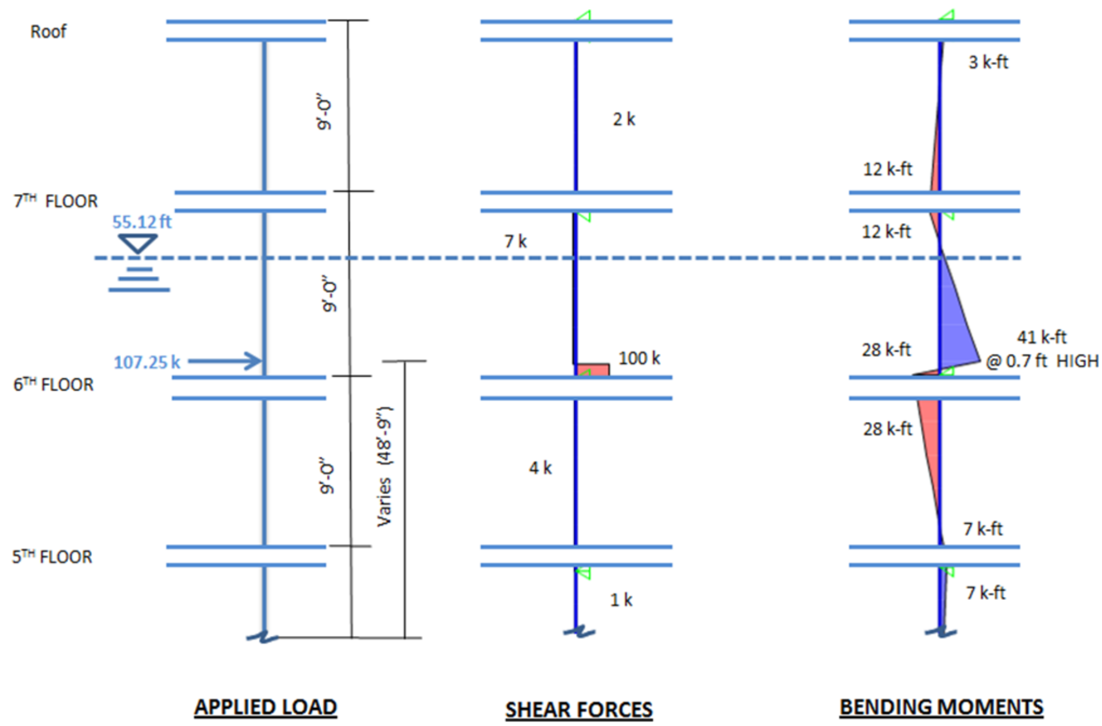


Figure C-129: Impact load applied at d away from the end of column on the 6th floor

Impact load at mid-height:

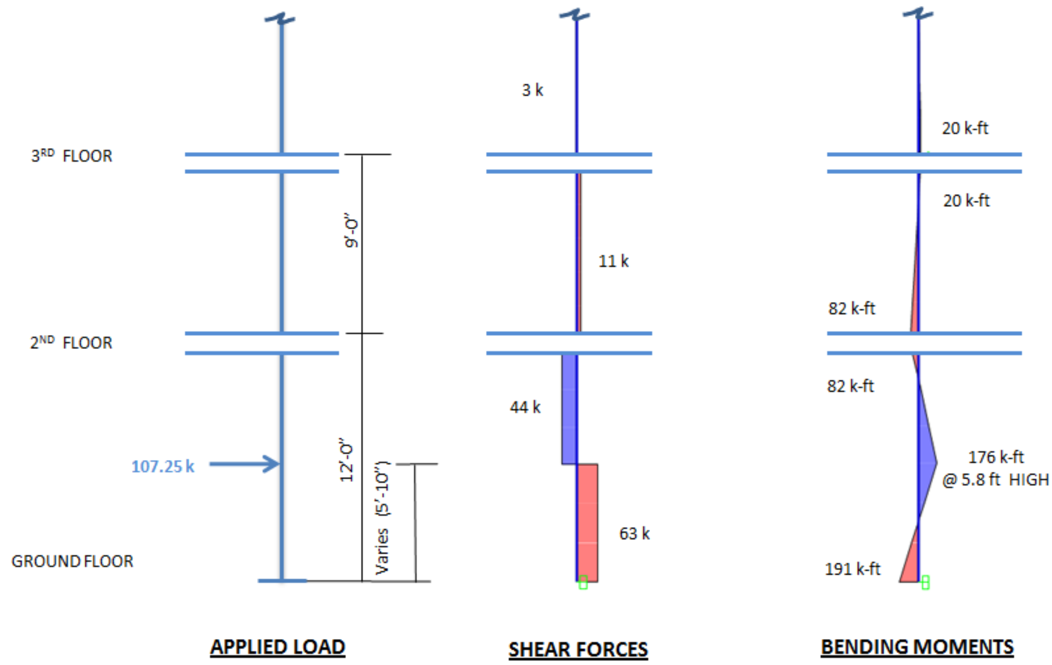


Figure C-130: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

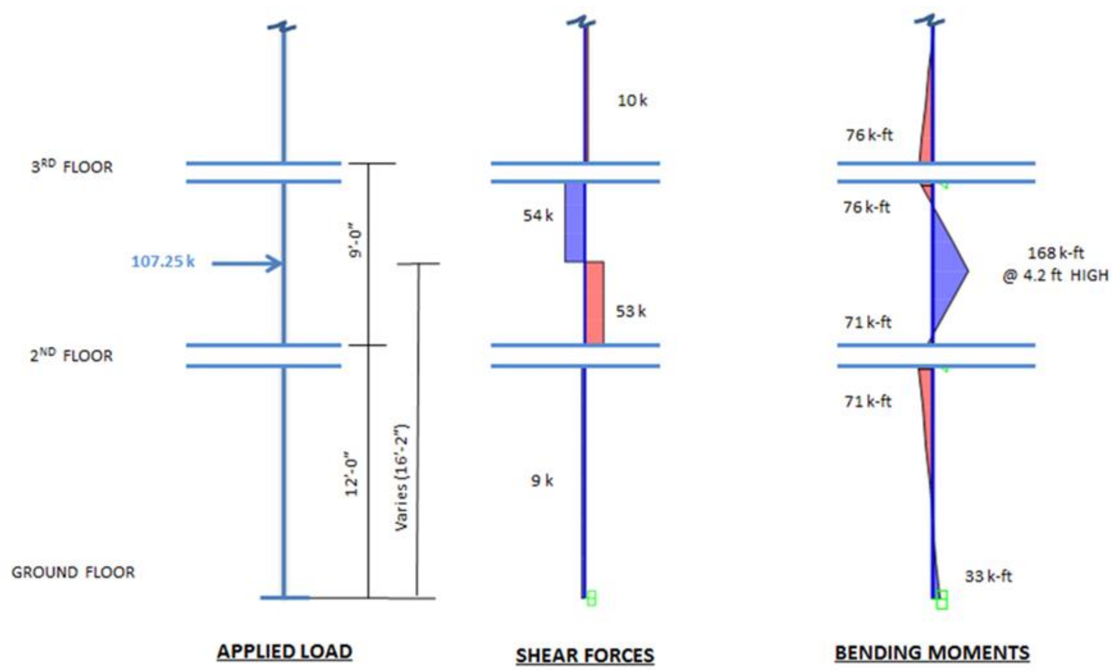


Figure C-131: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

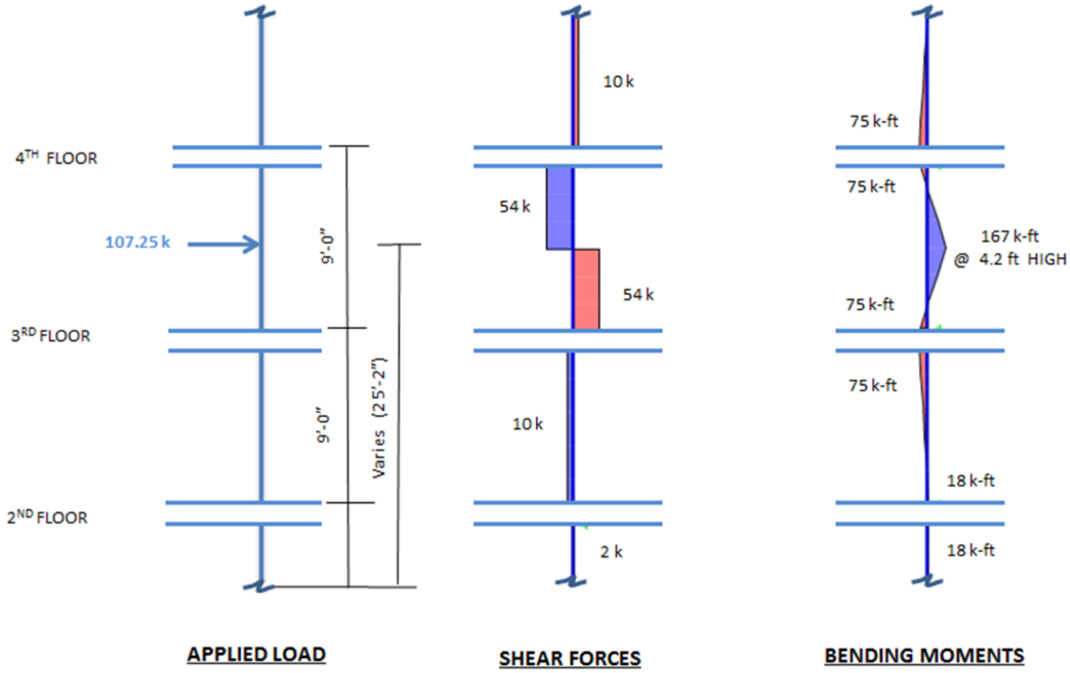


Figure C-132: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

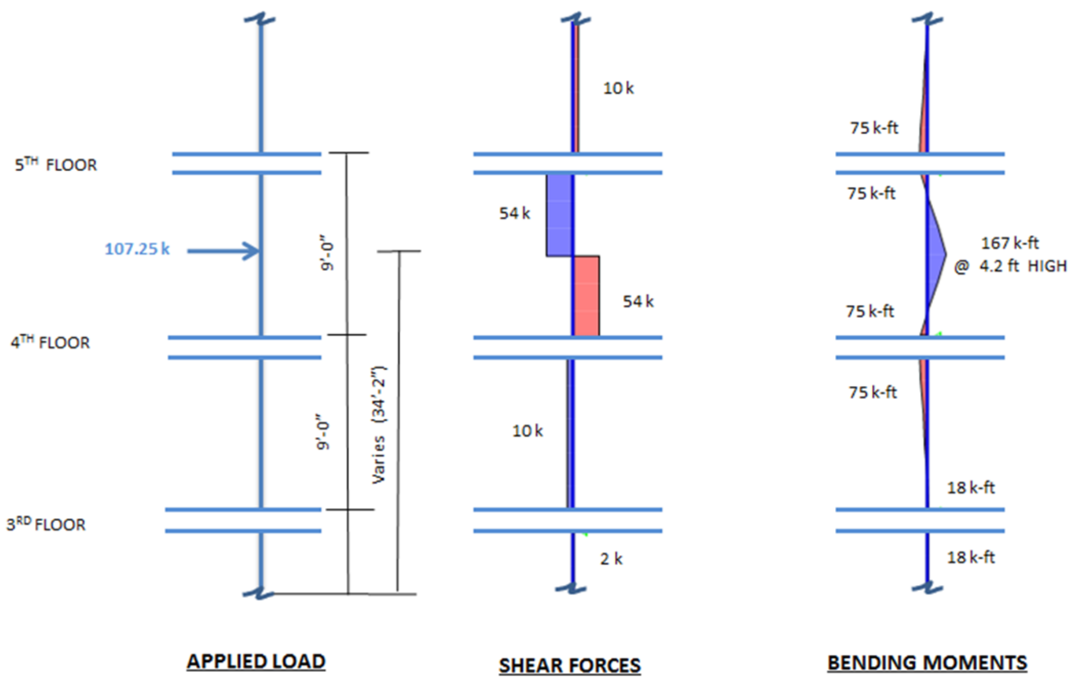


Figure C-133: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 4th floor column

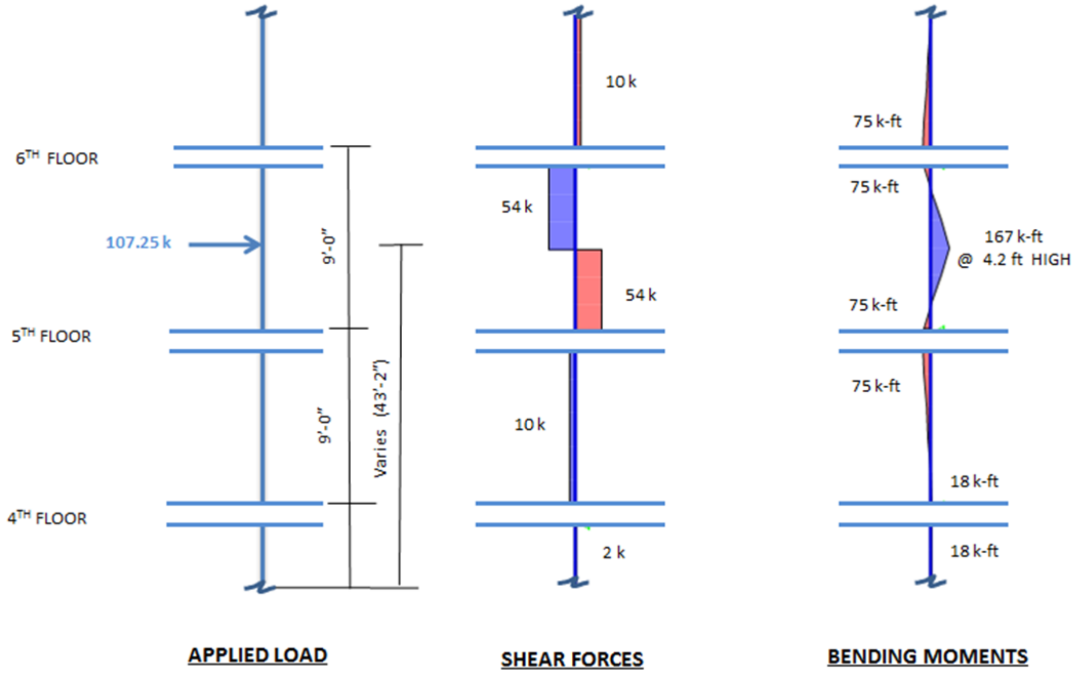


Figure C-134: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 5th floor column

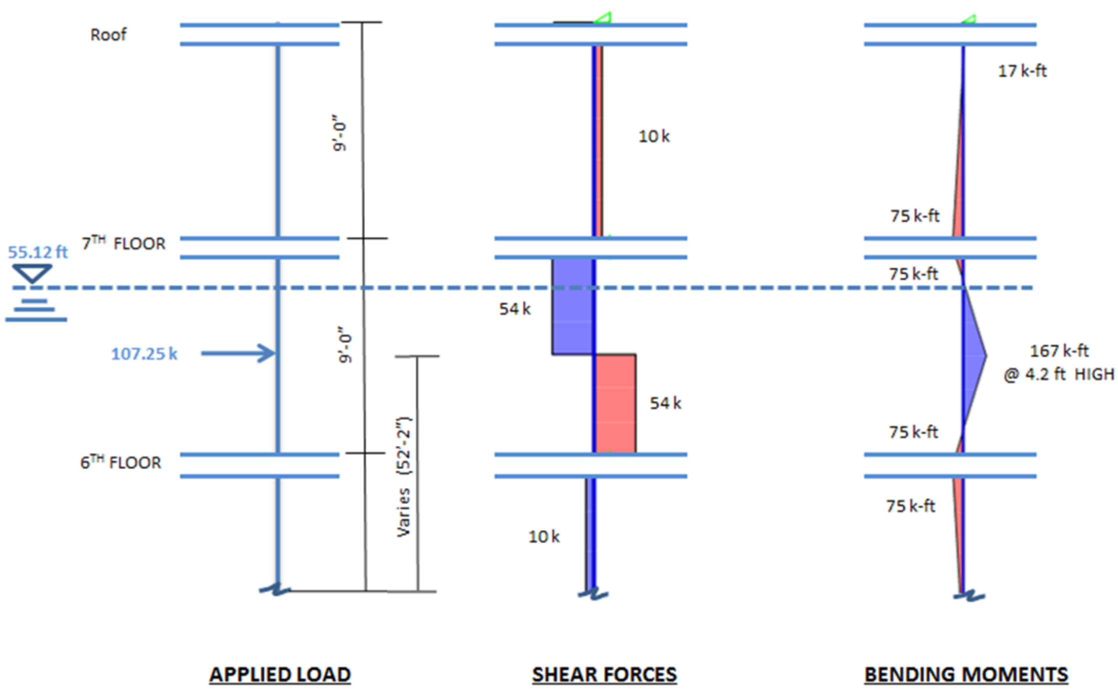


Figure C-135: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 6th floor column

Table C-8 summarizes the maximum critical load, bending moment and shear forces for all inundated shear walls using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro)

and long impact (Impact). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table C-8: Results from loading conditions of Hilo residential building exterior shear wall

Moment K-ft	Axial Load Kips	Shear @ d Kips	Loading Condition
Floor 1			
339	92.91	149	1.2D+Ftsu+0.5L (Hydro)
339	66.81	149	0.9D+Ftsu (Hydro)
176	92.91	102	1.2D+Ftsu+0.5L (Impact)
176	66.81	102	0.9D+Ftsu (Impact)
Floor 2			
252	79.63	105	1.2D+Ftsu+0.5L (Hydro)
252	57.27	105	0.9D+Ftsu (Hydro)
168	79.63	101	1.2D+Ftsu+0.5L (Impact)
168	57.27	101	0.9D+Ftsu (Impact)
Floor 3			
203	66.36	99	1.2D+Ftsu+0.5L (Hydro)
203	47.72	99	0.9D+Ftsu (Hydro)
167	66.36	101	1.2D+Ftsu+0.5L (Impact)
167	47.72	101	0.9D+Ftsu (Impact)
Floor 4			
203	53.09	104	1.2D+Ftsu+0.5L (Hydro)
203	38.18	104	0.9D+Ftsu (Hydro)
167	53.09	101	1.2D+Ftsu+0.5L (Impact)
167	38.18	101	0.9D+Ftsu (Impact)
Floor 5			
73	39.82	10	1.2D+Ftsu+0.5L (Hydro)
73	28.63	10	0.9D+Ftsu (Hydro)
167	39.82	101	1.2D+Ftsu+0.5L (Impact)
167	28.63	101	0.9D+Ftsu (Impact)
Floor 6			
17	26.54	2	1.2D+Ftsu+0.5L (Hydro)
17	19.09	2	0.9D+Ftsu (Hydro)
167	26.54	100	1.2D+Ftsu+0.5L (Impact)
167	19.09	100	0.9D+Ftsu (Impact)
Floor 7			
4	13.27	1	1.2D+Ftsu+0.5L (Hydro)
4	9.54	1	0.9D+Ftsu (Hydro)
75	13.27	10	1.2D+Ftsu+0.5L (Impact)
75	9.54	10	0.9D+Ftsu (Impact)

C.15.3.2 Existing Exterior Shear Wall Design for Combined Gravity and Tsunami Loads

The shear wall chosen at Grid Line D from **Figure C-17** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**.

Figure C-136 to **Figure C-142** shows the interaction diagram for the typical exterior shear wall including the tsunami load combinations.

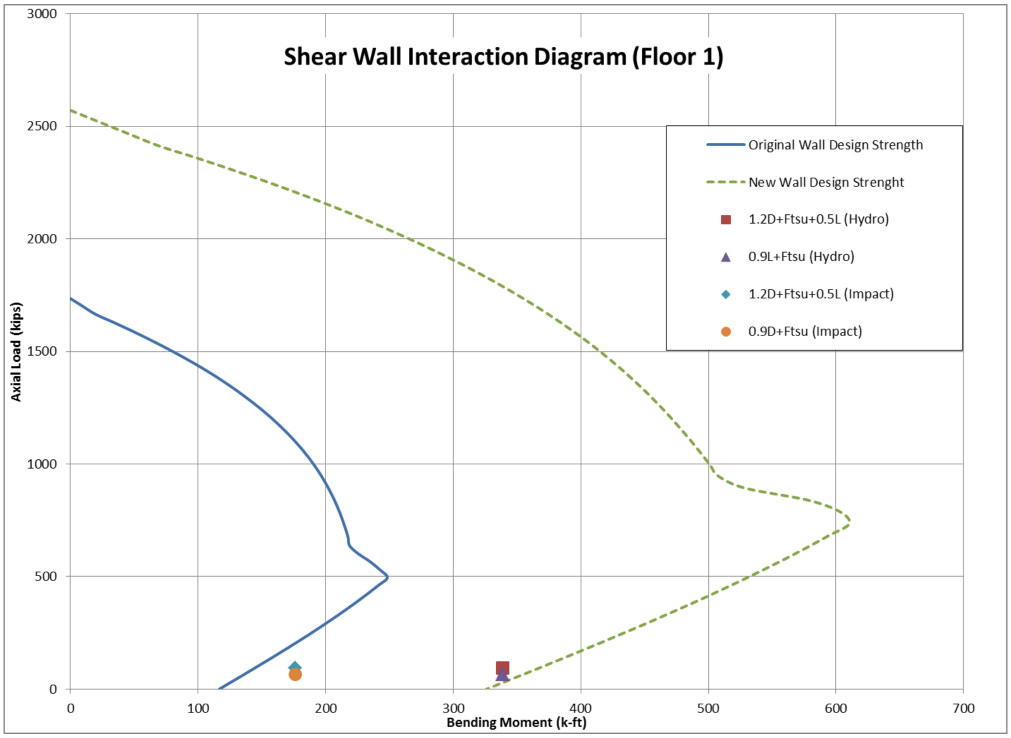


Figure C-136: Interaction diagram for typical ground floor exterior wall segment showing tsunami load combinations

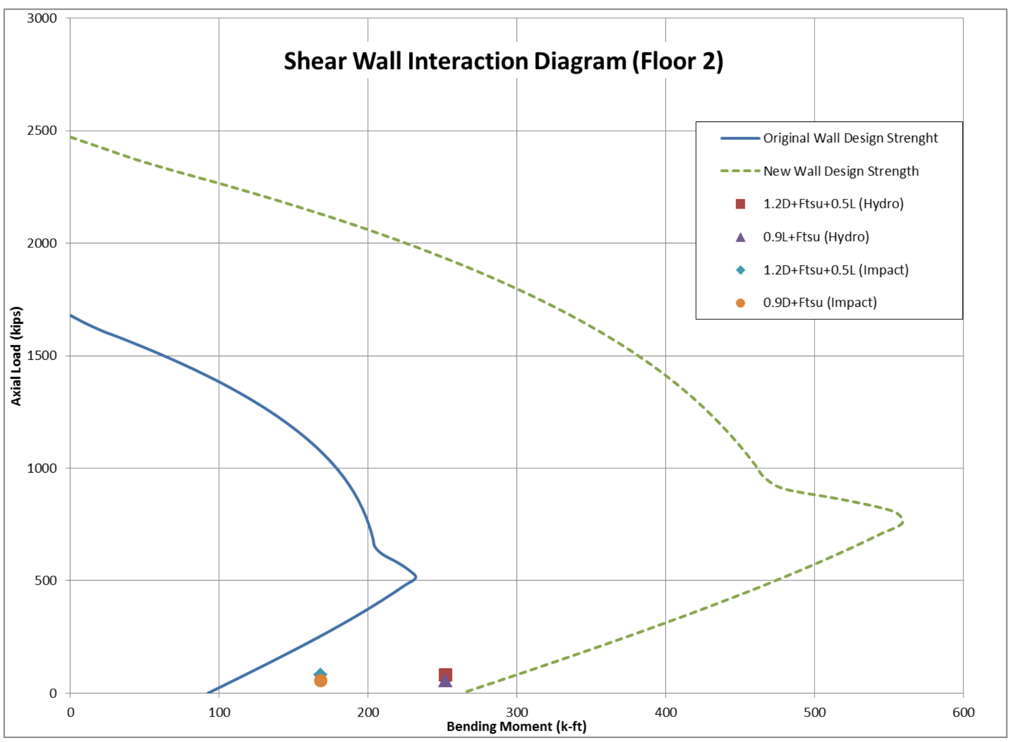


Figure C-137: Interaction diagram for typical 3rd floor exterior wall segment showing tsunami load combinations

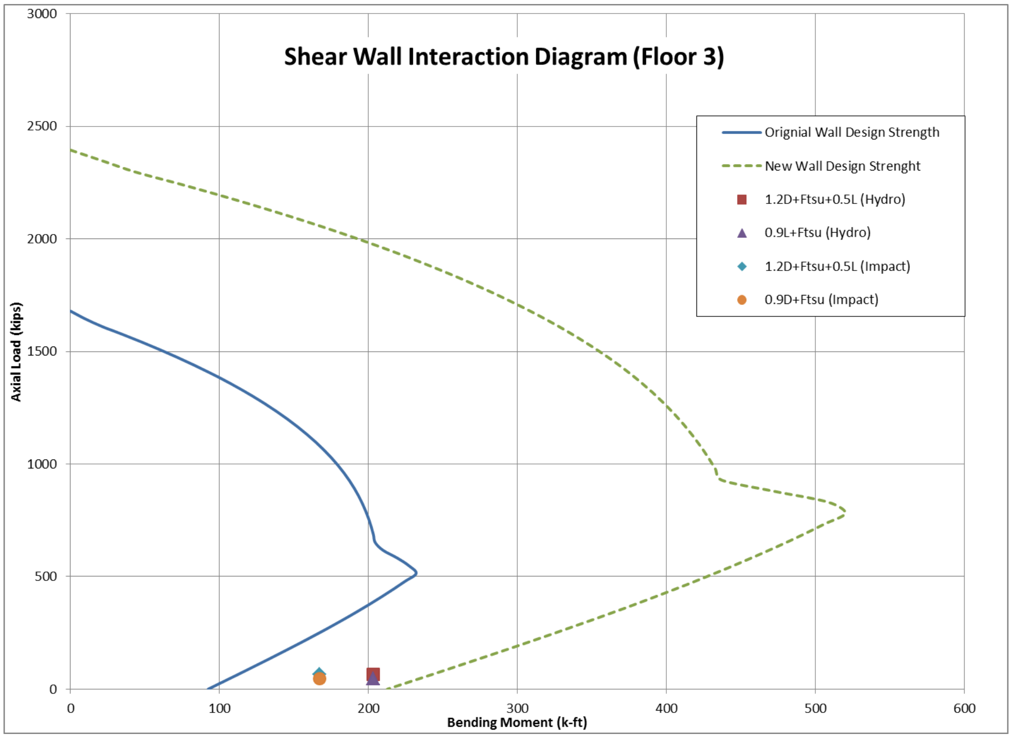


Figure C-138: Interaction diagram for typical 2nd floor exterior wall segment showing tsunami load combinations

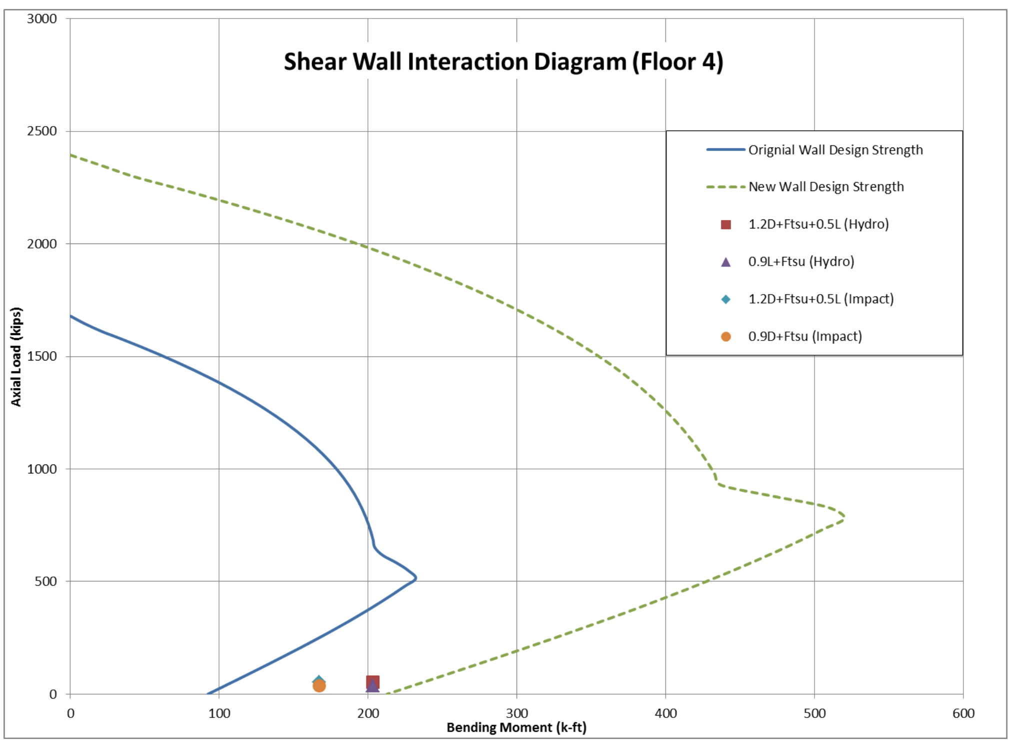


Figure C-139: Interaction diagram for typical 4th floor exterior wall segment showing tsunami load combinations

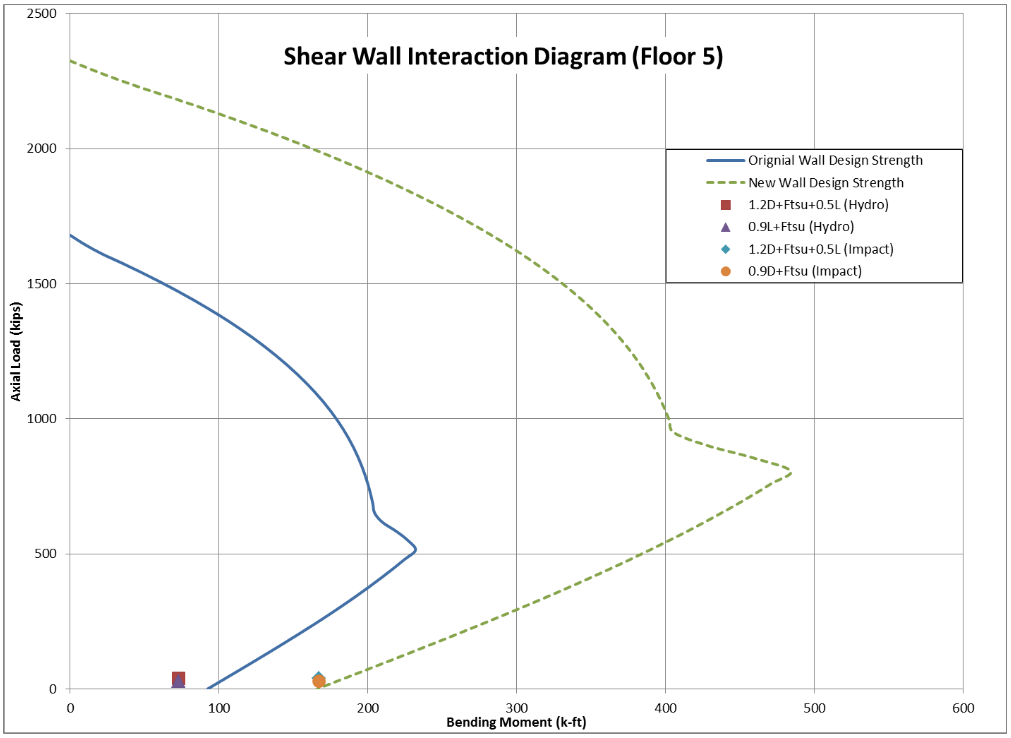


Figure C-140: Interaction diagram for typical 5th floor exterior wall segment showing tsunami load combinations

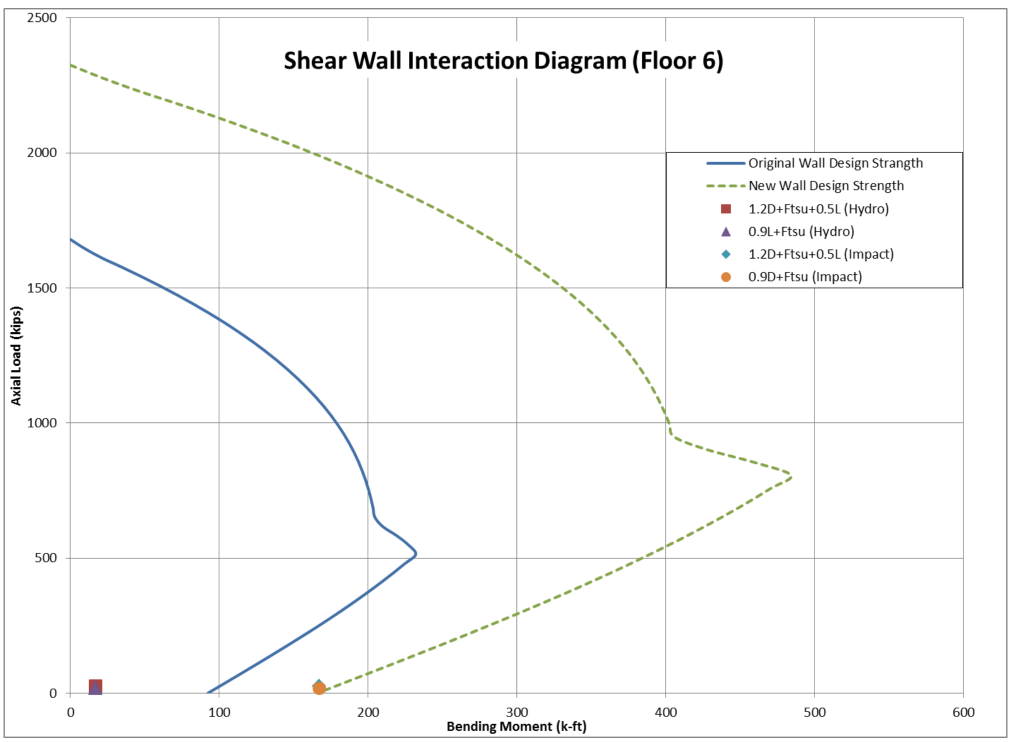


Figure C-141: Interaction diagram for typical 6th floor exterior wall segment showing tsunami load combinations

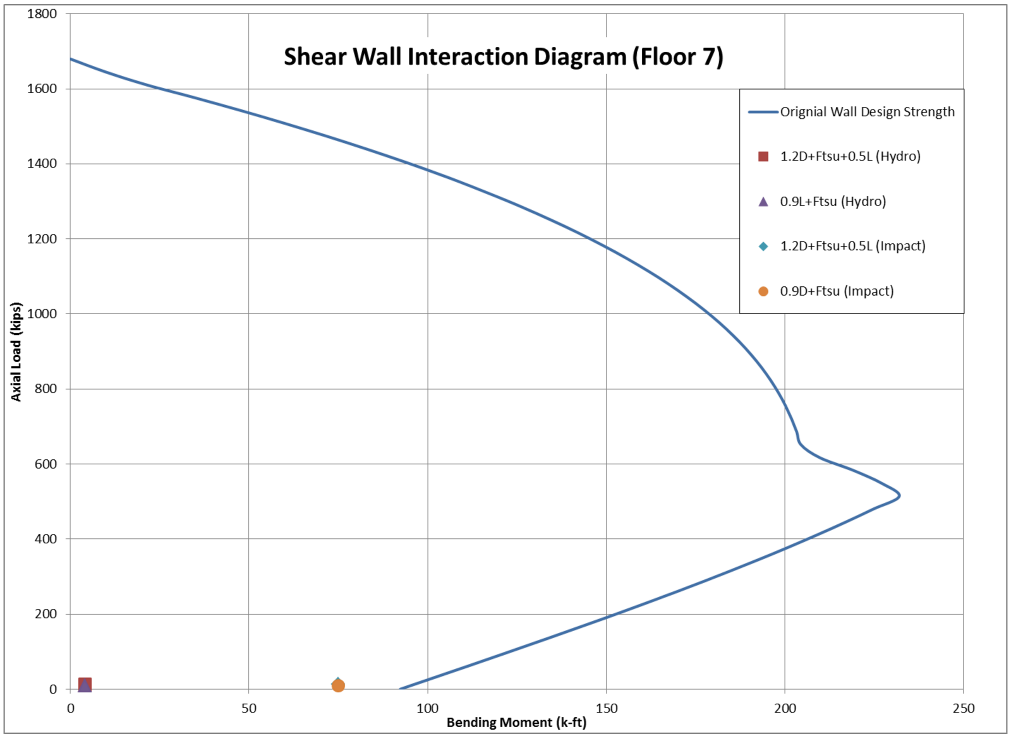


Figure C-142: Interaction diagram for typical 7th floor exterior wall segment showing tsunami load combinations

C.15.3.3 New Typical Shear Wall Design

The interaction diagrams show that the walls on floors 1 to 4 are inadequate for the bending moments due to hydrodynamic load, while those at levels 3 to 6 are inadequate for bending moments resulting from derbies impact. **Figure C-143** to **Figure C-146** show the revised wall designs required to resist the tsunami loads. **Figure C-147** and **Figure C-148** show the side view of the wall with shear stud rails included.

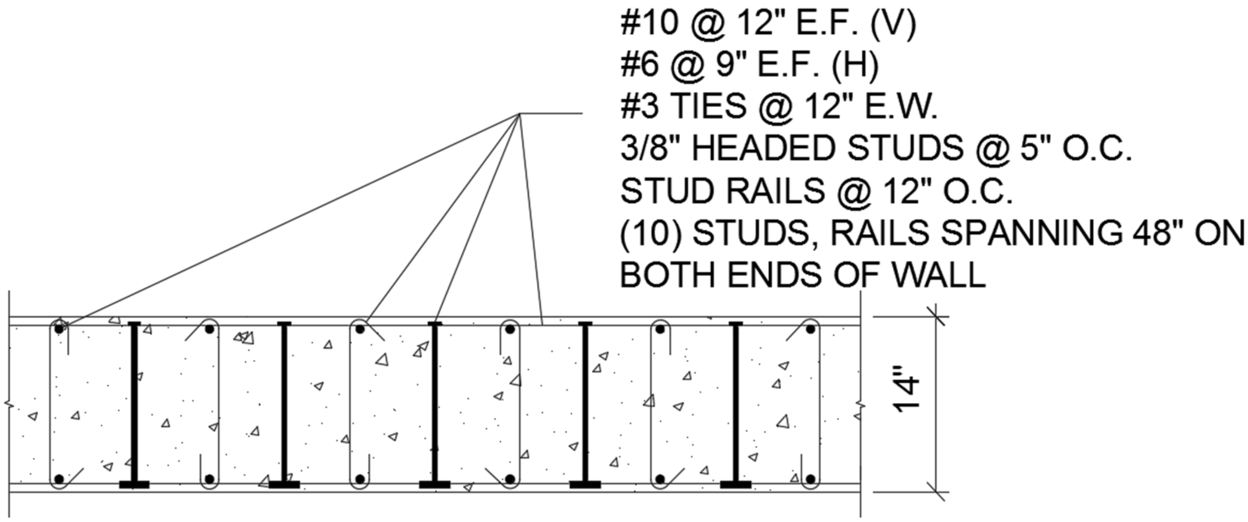


Figure C-143: New exterior wall, cross-section at the ground floor level based on tsunami design requirements.

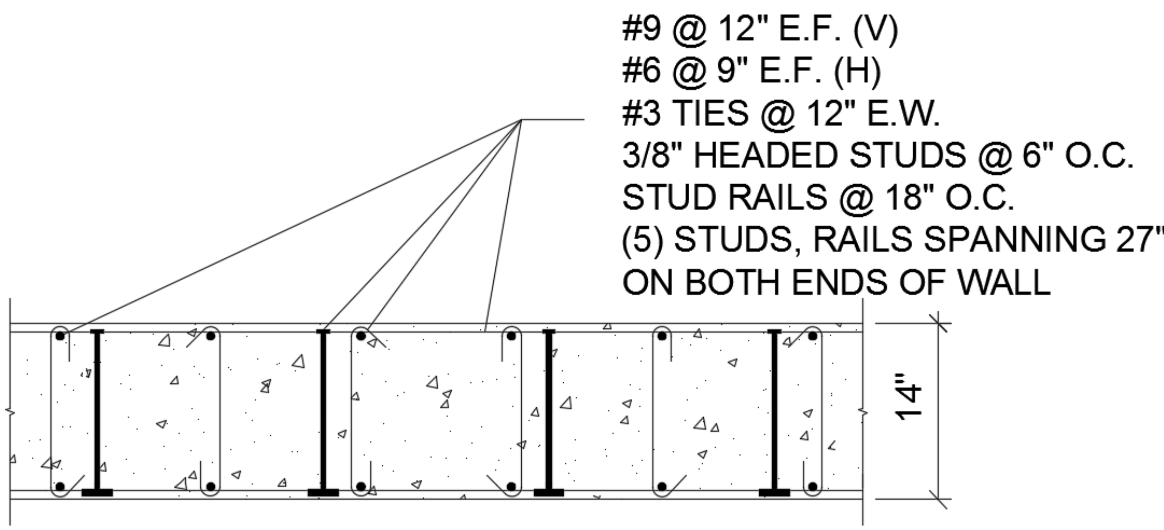


Figure C-144: New exterior wall, cross-section at the 2nd floor level based on tsunami design requirements.

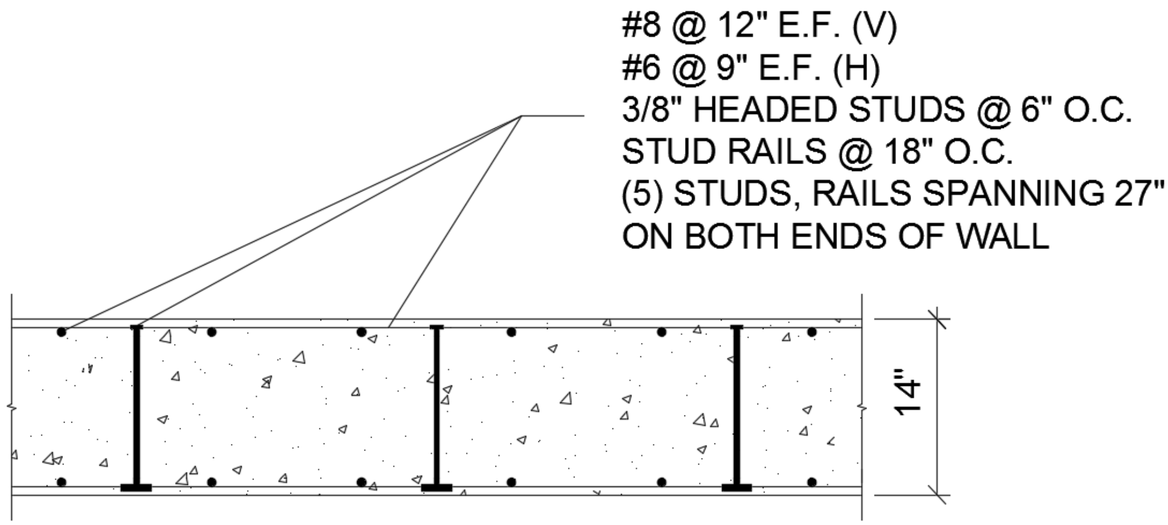


Figure C-145: New exterior wall, cross-section at the 2rd - 4th floor level based on tsunami design requirements.

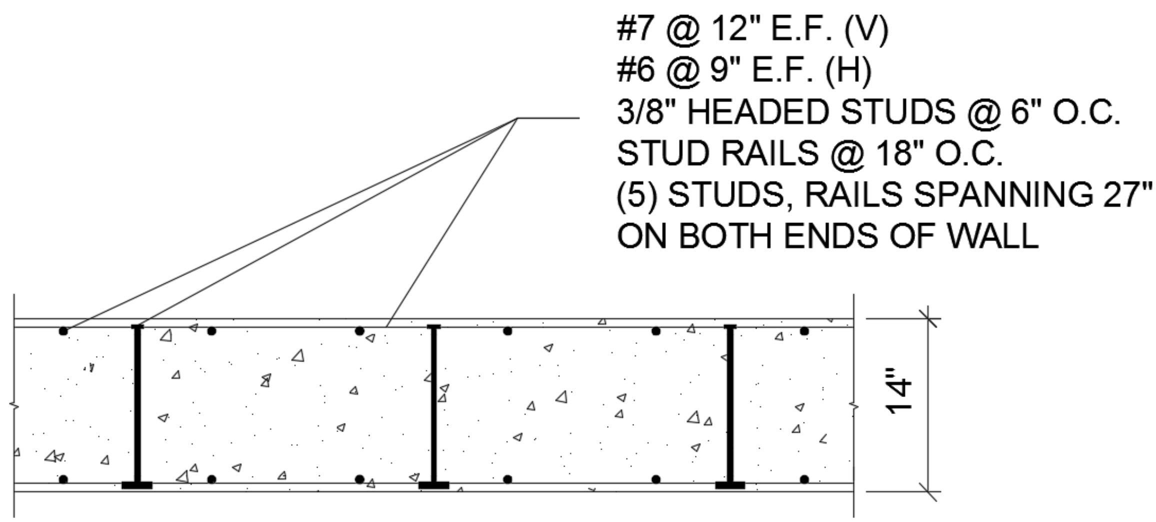


Figure C-146: New exterior wall, cross-section at the 5th - 6th floor level based on tsunami design requirements.

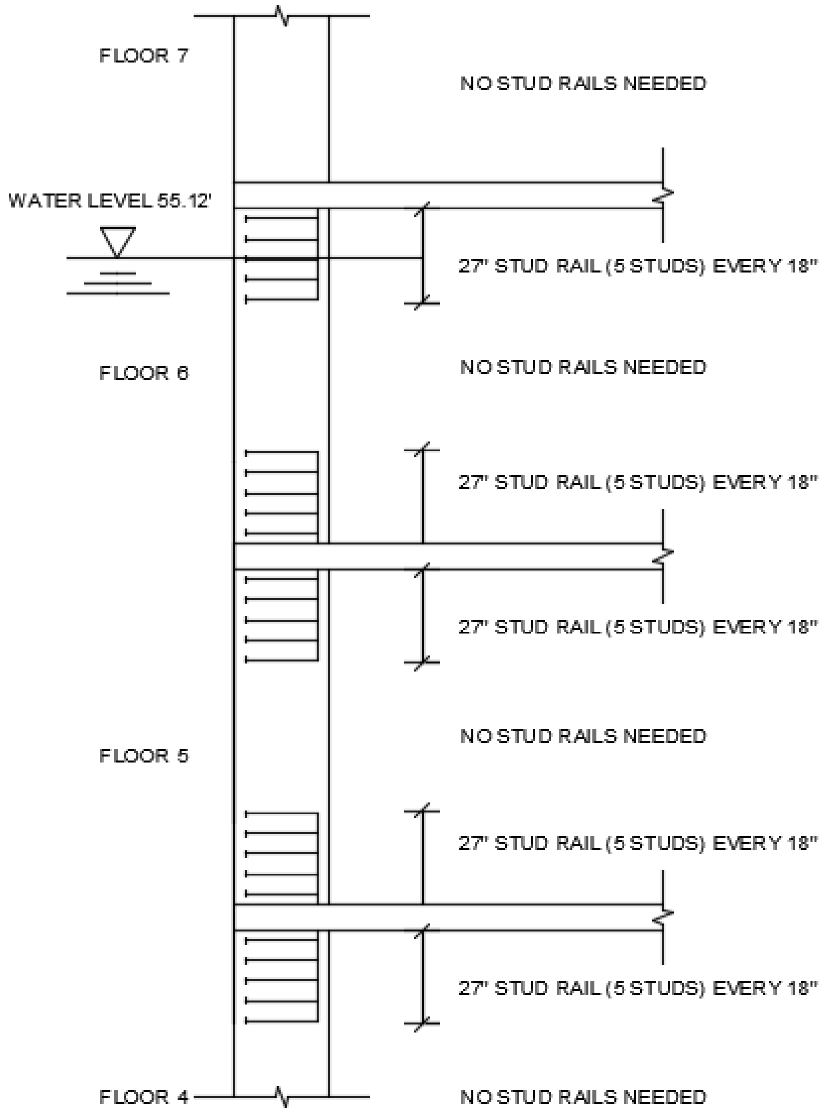


Figure C-147: Stud Rail Diagram for the Floor 4 – 7

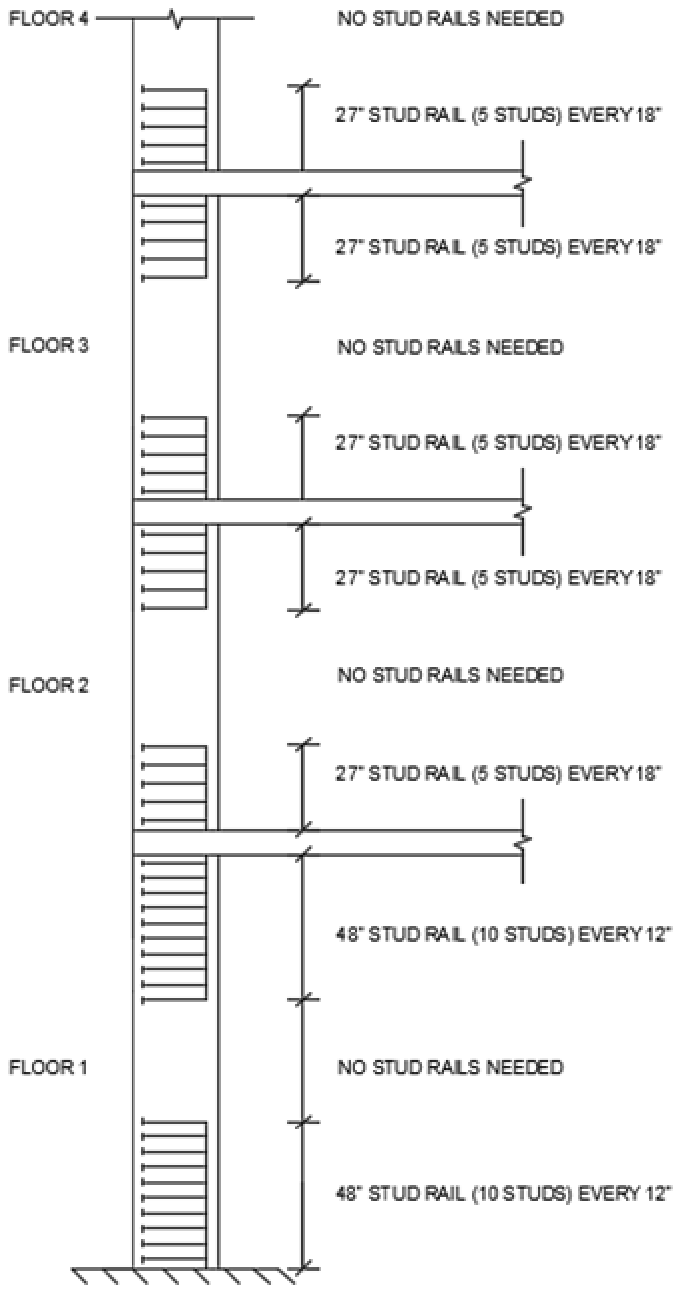


Figure C-148: Stud Rail Diagram for the Floor 1 – 4

C.15.3.4 Exterior Shear Wall Shear Design

Critical Shears:

1st Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$), P_u = 66.81 k:

Shear Capacity of existing shear wall (10" thick):

$\phi V_n = \phi(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\, b\, d = 2\left(1 + \frac{7{,}833}{2{,}000\times68\times10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.75/1{,}000 = 76$ kips

$N_u = (\frac{P_u\times1{,}000}{I_w}) \times b = \left(\frac{66.81\times1{,}000}{560}\right) \times 68 = 8{,}113\ lb$

I_w = 28'+11'+11'- 2 x 10"(thickness) = 580 in

$\phi V_n = \phi(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 149 kips > ϕV_n = 57 kips ∴ 92 kips needed

Shear Capacity for New Shear Wall (14" thick):

$\phi V_n = \phi(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\, b\, d = 2\left(1 + \frac{7{,}942}{2{,}000\times68\times14}\right)\ 1\sqrt{4000}\ \times 68 \times 12.75/1{,}000 = 110$ kips

$N_u = (\frac{P_u\times1{,}000}{I_w}) \times b = \left(\frac{66.81\times1{,}000}{572}\right) \times 68 = 7{,}942\ lb$

I_w = 28'+11'+11'- 2 x 14" (thickness) = 572 in

$\phi V_n = \phi(V_C + V_S)$ = 0.75 (110+0) = 83 kips

V_{tsu} = 149 kips > ϕV_n = 83 kips ∴ 89 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{5.67 \times 0.11 \times 60 \times 12.75}{5}$ = 96 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{12"} = 5.67$

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =12.75/2 = 6.375 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_{s\,(needed)}} = \frac{5.67 \times 0.11 \times 60 \times 12.75}{89}$ = 5.4 in

∴ s_{used} = 5 in

$\phi V_n = \phi(V_C + V_S)$ = 0.75 (110 + 96) = 155 Kips

ϕV_n = 155 Kips > V_{Tsu} = 149 Kips Therefore the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 48''}$ = 58 Kips ≤ $ØV_c$ = 83 Therefor rails go up 48" (10 Studs) at each end of the Shear Wall

2nd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 57.27 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{6{,}714}{2{,}000\times 68\times 10}\right)\,1\,\sqrt{4000}\,\times 68\times 8.8125/1{,}000$ = 76 kips

$N_u = \left(\frac{P_u\times 1{,}000}{I_w}\right)\times b = \left(\frac{57.27\times 1{,}000}{580}\right)\times 68 = 6{,}714\ lb$

I_w = 28'+11'+11'- 2 x 10"(thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 105 kips > $ØV_n$ = 57 kips ∴ 64 kips needed

Shear Capacity for New Shear Wall (14" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{6{,}808}{2{,}000\times 68\times 14}\right)\,1\,\sqrt{4000}\,\times 68\times 12.875/1{,}000$ = 111 kips

$N_u = \left(\frac{P_u\times 1{,}000}{I_w}\right)\times b = \left(\frac{57.27\times 1{,}000}{572}\right)\times 68 = 6{,}808\ lb$

I_w = 28'+11'+11'- 2 x 14" (thickness) = 572 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (111+0) = 83 kips

V_{tsu} = 105 kips > $ØV_n$ = 83 kips ∴ 29 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n\times A_v\times f_y\times d}{s} = \frac{3.78\times 0.11\times 60\times 12.875}{6}$ = 54 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{18"} = 3.78$

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =12.875/2 = 6.44 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{29}$ = 11.2 in

∴ s_{used} = 6 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (111 + 54) = 124 Kips

$ØV_n$ = 124 Kips > V_{Tsu} = 105 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 27''}$ = 81 Kips < $ØV_c$ = 83 Therefore rails go up 27" (5 Studs) at each end of the Shear Wall

3rd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 47.72 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{6{,}714}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000$ = 76 kips

$N_u = \left(\frac{P_u \times 1{,}000}{I_w}\right) \times b = \left(\frac{47.72 \times 1{,}000}{580}\right) \times 68 = 6{,}714\ lb$

I_w = 28'+11'+11'- 2 x 10"(thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 101 kips > $ØV_n$ = 57 kips ∴ 64 kips needed

Shear Capacity for New Shear Wall (14" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{6{,}808}{2{,}000 \times 68 \times 14}\right)\ 1\sqrt{4000}\ \times 68 \times 12.875/1{,}000$ = 111 kips

$N_u = \left(\frac{P_u \times 1{,}000}{I_w}\right) \times b = \left(\frac{47.72 \times 1{,}000}{572}\right) \times 68 = 6{,}808\ lb$

I_w = 28'+11'+11'- 2 x 14" (thickness) = 572 in

$ØV_n = Ø(V_C + V_S)$ = .75 (111+0) = 83 kips

V_{tsu} = 101 kips > $ØV_n$ = 83 kips ∴ 29 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{6}$ = 54 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{18"}$ = 3.78

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =12.875/2 = 6.44 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{24}$ = 13.7 in

∴ s_{used} = 6 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (111 + 54) = 124 Kips

$ØV_n$ = 124 Kips > V_{Tsu} = 101 Kips Therefore the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 27''}$ = 81 Kips < $ØV_c$ = 83 Therefor rails go up 27" (5 Studs) at each end of the Shear Wall

4th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 38.18 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{4{,}476}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000$ = 77 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{38.18 \times 1{,}000}{560}\right) \times 68 = 4{,}476\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (77+0) = 58 kips

V_{tsu} = 104 kips > $ØV_n$ = 58 kips ∴ 62 kips needed

Shear Capacity for New Shear Wall (14" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{4{,}539}{2{,}000 \times 68 \times 14}\right)\ 1\sqrt{4000}\ \times 68 \times 12.875/1{,}000$ = 111 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{38.18 \times 1{,}000}{572}\right) \times 68 = 4{,}539\ lb$

l_w = 28'+11'+11'- 2 x 14" (thickness) = 572 in

$ØV_n = Ø(V_C + V_S)$ = .75 (111+0) = 83 kips

V_{tsu} = 104 kips > $ØV_n$ = 83 kips ∴ 28 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{6}$ = 54 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{18"}$ = 3.78

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =12.875/2 = 6.44 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{28}$ = 11.7 in

∴ s_{used} = 6 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (111 + 54) = 124 Kips

$ØV_n$ = 124 Kips > V_{Tsu} = 104 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 27''}$ = 81 Kips < $ØV_c$ = 83 Therefore rails go up 27" (5 Studs) at each end of the Shear Wall

5th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 28.63 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{3,357}{2,000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1,000$ = 77 kips

$N_u = (\frac{P_u \times 1,000}{l_w}) \times b = \left(\frac{28.63 \times 1,000}{580}\right) \times 68 = 3,357\ lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (77+0) = 58 kips

V_{tsu} = 101 kips > $ØV_n$ = 58 kips ∴ 58 kips needed

Shear Capacity for New Shear Wall (14" thick):

$\varnothing V_n = \varnothing(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{3{,}404}{2{,}000\times68\times14}\right)\,1\sqrt{4000}\,\times 68\times 12.875/1{,}000$ = 111 kips

$N_u = \left(\frac{P_u\times1{,}000}{I_w}\right)\times b = \left(\frac{28.63\times1{,}000}{572}\right)\times 68 = 3{,}404\ lb$

I_w = 28'+11'+11'- 2 x 14" (thickness) = 572 in

$\varnothing V_n = \varnothing(V_C + V_S)$ = 0.75 (111+0) = 83 kips

V_{tsu} = 101 kips > $\varnothing V_n$ = 83 kips ∴ 24 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n\times A_v\times f_y\times d}{s} = \frac{3.78\times0.11\times60\times12.875}{6}$ = 54 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{18"}$ = 3.78

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =12.875/2 = 6.44 in

$s_{needed} = \frac{n\times A_v\times f_y\times d}{V_S} = \frac{3.78\times0.11\times60\times12.875}{24}$ = 13.6 in

∴ s_{used} = 6 in

$\varnothing V_n = \varnothing(V_C + V_S)$ = 0.75 (111 + 54) = 124 Kips

$\varnothing V_n$ = 124 Kips > V_{Tsu} = 104 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 27''}$ = 81 Kips < $\varnothing V_c$ = 83 Therefore rails go up 27" (5 Studs) at each end of the Shear Wall

6th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 19.09 k:

Shear Capacity of existing shear wall (10" thick):

$\varnothing V_n = \varnothing(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{2{,}238}{2{,}000\times68\times10}\right)\,1\sqrt{4000}\,\times 68\times 8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u \times 1{,}000}{l_w}) \times b = \left(\frac{19.09 \times 1{,}000}{580}\right) \times 68 = 2{,}238\ lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$\emptyset V_n = \emptyset(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 100 kips > $\emptyset V_n$ = 57 kips ∴ 57 kips needed

Shear Capacity for New Shear Wall (14" thick):

$\emptyset V_n = \emptyset(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\ b\ d = 2\left(1 + \frac{2{,}269}{2{,}000 \times 68 \times 14}\right)\ 1\sqrt{4000}\ \times 68 \times 12.875/1{,}000$ = 111 kips

$N_u = (\frac{P_u \times 1{,}000}{l_w}) \times b = \left(\frac{19.09 \times 1{,}000}{544}\right) \times 68 = 2{,}269 lb$

l_w = 28'+11'+11'- 2 x 14" (thickness) = 572 in

$\emptyset V_n = \emptyset(V_C + V_S)$ = 0.75 (111+0) = 83 kips

V_{tsu} = 100 kips > $\emptyset V_n$ = 83 kips ∴ 22 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{6}$ = 54 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{18"} = 3.78$

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =12.875/2 = 6.44 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{22}$ = 14.4 in

∴ s_{used} = 6 in

$\emptyset V_n = \emptyset(V_C + V_S)$ = 0.75 (111 + 54) = 124 Kips

$\emptyset V_n$ = 124 Kips > V_{Tsu} = 100 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 27''}$ = 81 Kips < $\emptyset V_c$ = 83 Therefore rails go up 27" (5 Studs) at each end of the Shear Wall

7th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9D + F_{TSU}), P_u = 9.54 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1+\frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1+\frac{1{,}119}{2{,}000\times68\times10}\right)\,1\sqrt{4000}\,\times 68\times 8.875/1{,}000$ = 76 kips

$N_u = \left(\frac{P_u\times1{,}000}{I_w}\right)\times b = \left(\frac{9.54\times1{,}000}{580}\right)\times 68 = 1{,}119\ lb$

I_w = 28'+11'+11'- 4 x 10" (thickness) = 560 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 10 kips < $ØV_n$ = 57 kips ∴ no shear studs ae needed

C.15.3.5 Overall Wall loading:

Floor 1 Try 1 (Elevator):

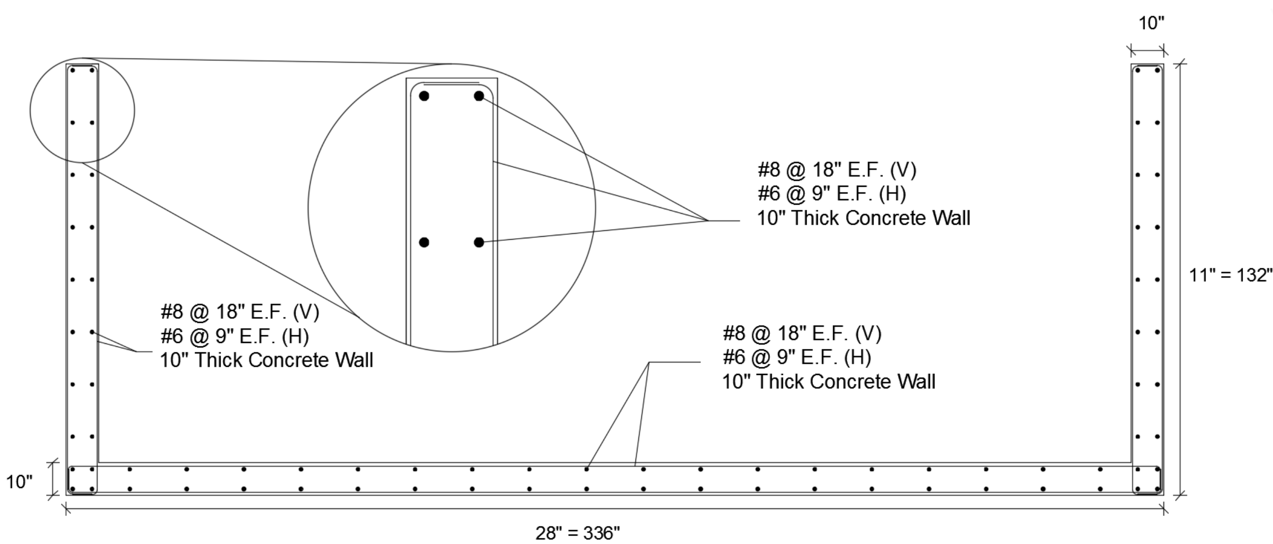


Figure C-149: Original Elevator/ Mech. Room shear wall cross-section at the ground floor level

Floor 2 Try 1 (Elevator):

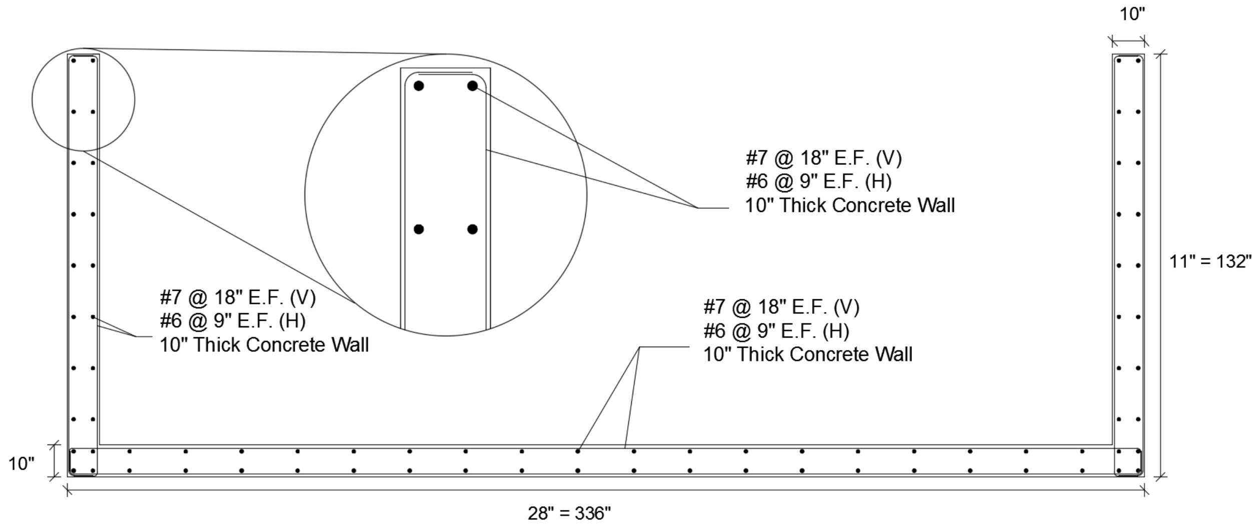


Figure C-150: Original Elevator/ Mech. Room shear wall cross-section at the 2nd floor level

Floor 3-7 Try 1 (Elevator):

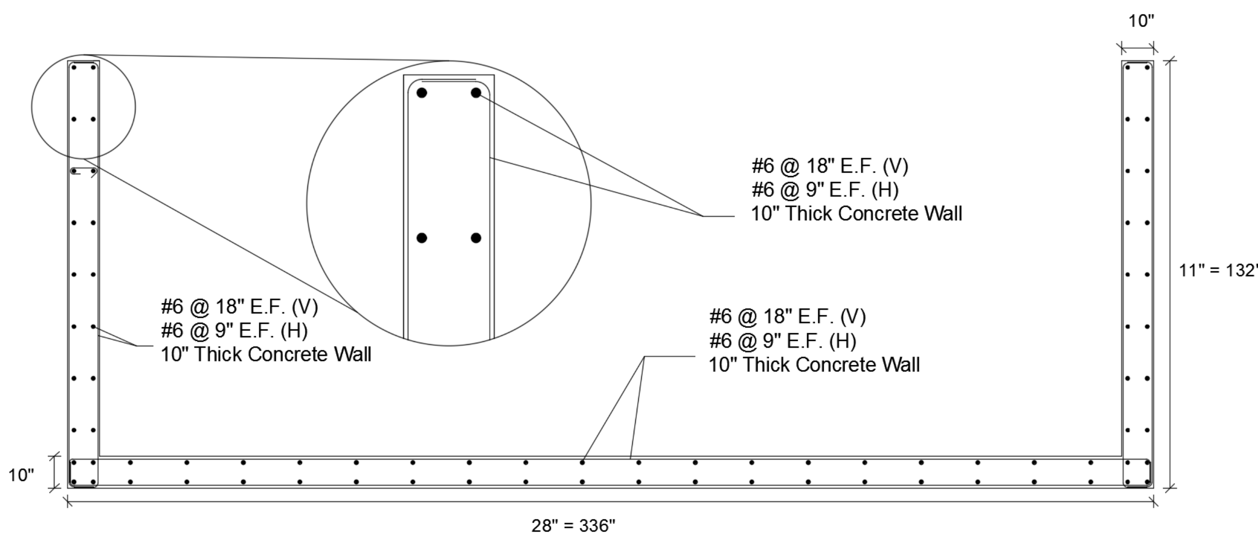


Figure C-151: Original Elevator/ Mech. Room shear wall cross-section at the 3rd floor level

Floor 1 Try 1 (Stairs):

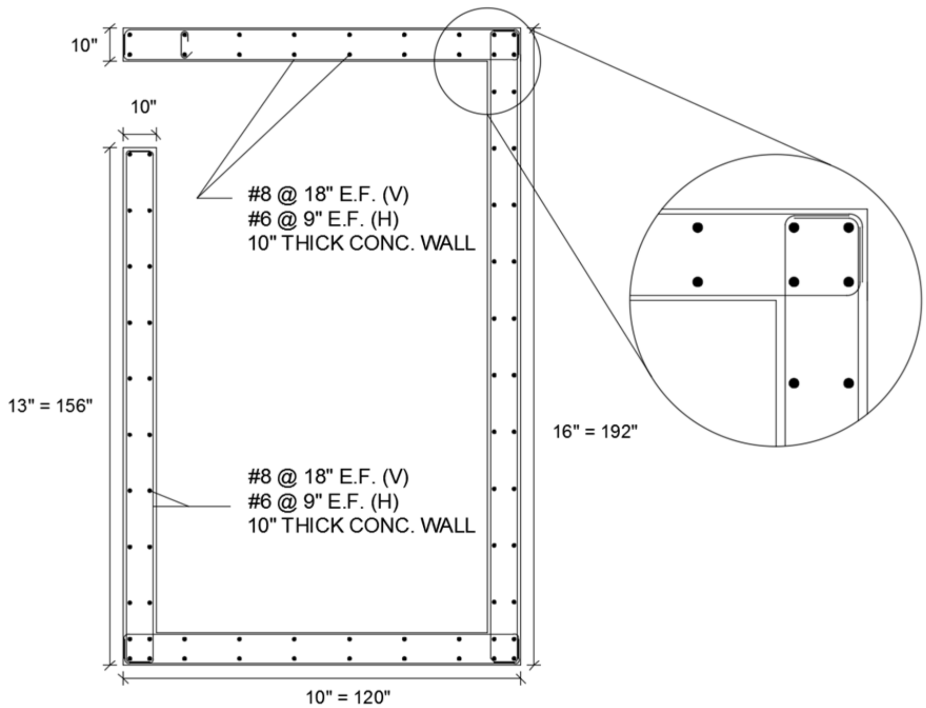


Figure C-152: Original stairwell shear wall cross-section at the ground floor level

Floor 2 Try 1 (Stairs):

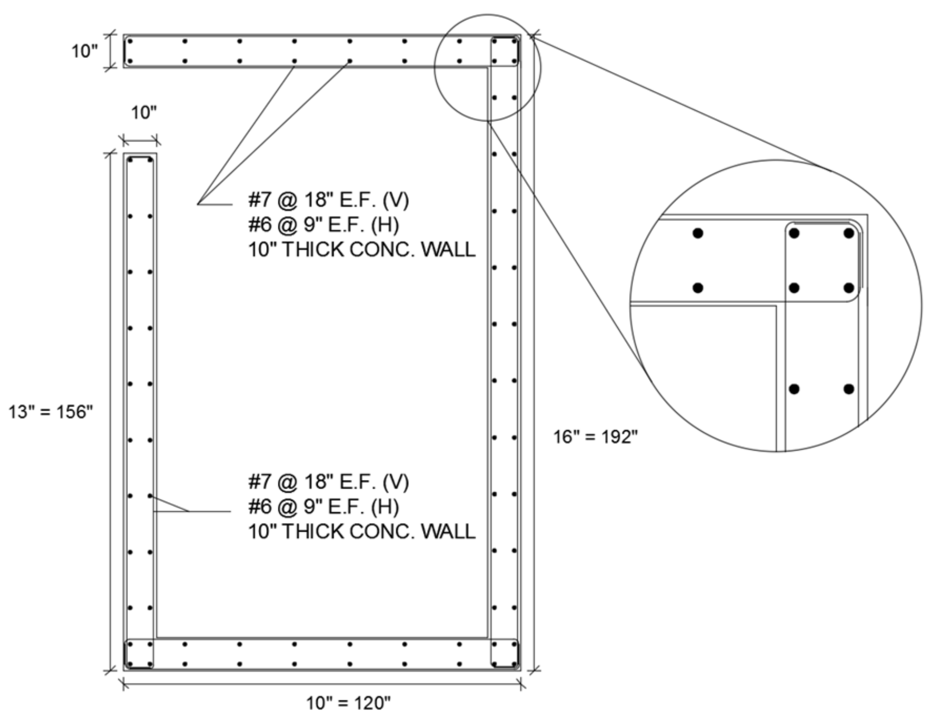


Figure C-153: Original stairwell shear wall cross-section at the 2nd floor level

Floor 3 Try 1 (Stairs):

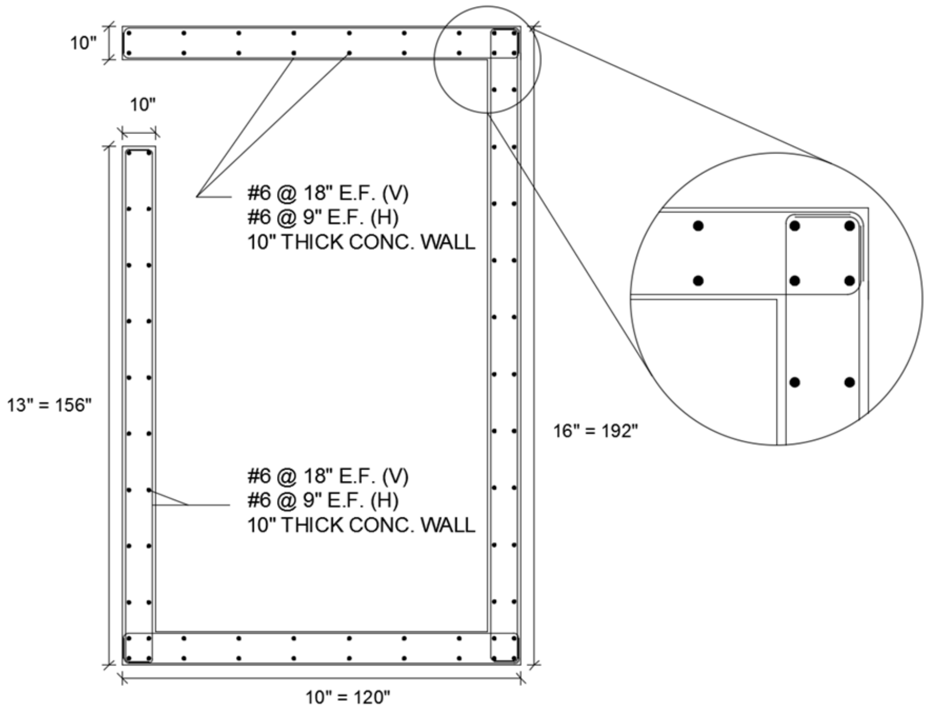


Figure C-154: Original stairwell shear wall cross-section at the 3rd floor level

C.15.3.6 Gravity Load Calculation (for completeness)

The gravity load tributary width is 5.5 ft. Gravity loads will be computed per foot width of wall.

The Dead Load at the base of the wall is:

$P_D = [128(5.5)(1)(6)+123(5.5)(1)+0.83(1)(150)(66)]/1000 = 13.1$ k/ft

Floor Live load reduction: Reduction Factor = $0.25+15/[1(5.5)(28)(6)]^{0.5} = 0.743$

$P_L = 0.743[55(5.5)(1)(6)]/1000 = 1.35$ k/ft

Roof Live Load reduction: Reduction Factor = $R_1R_2= [1.2-(0.001)(5.5)(28)](1.0) = 1.05$*, Use 1.0*

$P_{Lr} = 20(5.5)(1)/1000 = 0.110$ k/ft

Analysis of a 50 foot width of wall with the applied tsunami hydrodynamic loads from Load Case 2 results in the following bending moment and shear force diagrams:

Table C-9 and **Table C-10** summarizes the maximum critical load, bending moment and shear forces for all inundated shear walls using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and long impact (Impact). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table C-9: Results from loading conditions of Hilo residential building Overall shear walls (Floor 1 - 2)

Moment k-ft	Axial Load Kips	Shear Kips	Load Combination
Floor 1			
Earthquake			
5,753	630	161	1.2D+Ftsu+0.5L (Elev/Mech)
5,753	399	161	0.9D+Ftsu (Eelv/Mech)
-5,753	630	161	1.2D+Ftsu+0.5L (Elev/Mech)
-5,753	399	161	0.9D+Ftsu (Elev/Mech)
16,390	796	340	1.2D+Ftsu+0.5L (Stairs)
16,390	566	340	0.9D+Ftsu (Stairs)
-16,390	796	340	1.2D+Ftsu+0.5L (Stairs)
-16,390	566	340	0.9D+Ftsu (Stairs)
Tsunami			
20,817	132	581	1.2D+Ftsu+0.5L (Elev/Mech)
20,817	-99	581	0.9D+Ftsu (Eelv/Mech)
-20,817	132	581	1.2D+Ftsu+0.5L (Elev/Mech)
-20,817	-99	581	0.9D+Ftsu (Elev/Mech)
59,307	735	1,230	1.2D+Ftsu+0.5L (Stairs)
59,307	505	1,230	0.9D+Ftsu (Stairs)
-59,307	735	1,230	1.2D+Ftsu+0.5L (Stairs)
-59,307	505	1,230	0.9D+Ftsu (Stairs)
Floor 2			
Earthquake			
3,930	539	138	1.2D+Ftsu+0.5L (Elev/Mech)
3,930	342	138	0.9D+Ftsu (Eelv/Mech)
-3,930	539	138	1.2D+Ftsu+0.5L (Elev/Mech)
-3,930	342	138	0.9D+Ftsu (Elev/Mech)
12,310	686	337	1.2D+Ftsu+0.5L (Stairs)
12,310	489	337	0.9D+Ftsu (Stairs)
-12,310	686	337	1.2D+Ftsu+0.5L (Stairs)
-12,310	489	337	0.9D+Ftsu (Stairs)
Tsunami			
14,224	112	500	1.2D+Ftsu+0.5L (Elev/Mech)
14,224	-86	500	0.9D+Ftsu (Eelv/Mech)
-14,224	112	500	1.2D+Ftsu+0.5L (Elev/Mech)
-14,224	-86	500	0.9D+Ftsu (Elev/Mech)
44,543	643	1,219	1.2D+Ftsu+0.5L (Stairs)
44,543	445	1,219	0.9D+Ftsu (Stairs)
-44,543	643	1,219	1.2D+Ftsu+0.5L (Stairs)
-44,543	445	1,219	0.9D+Ftsu (Stairs)

Table C-10: Results from loading conditions of Hilo residential building Overall shear walls (Floor 3 - 4)

Moment k-ft	Axial Load Kips	Shear Kips	Load Combination
Floor 3			
Earthquake			
2,853	452	125	1.2D+Ftsu+0.5L (Elev/Mech)
2,853	288	125	0.9D+Ftsu (Eelv/Mech)
-2,853	452	125	1.2D+Ftsu+0.5L (Elev/Mech)
-2,853	288	125	0.9D+Ftsu (Elev/Mech)
9,278	575	311	1.2D+Ftsu+0.5L (Stairs)
9,278	411	311	0.9D+Ftsu (Stairs)
-9,278	575	311	1.2D+Ftsu+0.5L (Stairs)
-9,278	411	311	0.9D+Ftsu (Stairs)
Tsunami			
10,322	104	453	1.2D+Ftsu+0.5L (Elev/Mech)
10,322	-60	453	0.9D+Ftsu (Eelv/Mech)
-10,322	104	453	1.2D+Ftsu+0.5L (Elev/Mech)
-10,322	-60	453	0.9D+Ftsu (Elev/Mech)
33,574	549	1,124	1.2D+Ftsu+0.5L (Stairs)
33,574	385	1,124	0.9D+Ftsu (Stairs)
-33,574	549	1,124	1.2D+Ftsu+0.5L (Stairs)
-33,574	385	1,124	0.9D+Ftsu (Stairs)
Floor 4			
Earthquake			
1,932	365	111	1.2D+Ftsu+0.5L (Elev/Mech)
1,932	233	111	0.9D+Ftsu (Eelv/Mech)
-1,932	365	111	1.2D+Ftsu+0.5L (Elev/Mech)
-1,932	233	111	0.9D+Ftsu (Elev/Mech)
6,483	463	270	1.2D+Ftsu+0.5L (Stairs)
6,483	332	270	0.9D+Ftsu (Stairs)
-6,483	463	270	1.2D+Ftsu+0.5L (Stairs)
-6,483	332	270	0.9D+Ftsu (Stairs)
Tsunami			
6,990	94	403	1.2D+Ftsu+0.5L (Elev/Mech)
6,990	-38	403	0.9D+Ftsu (Eelv/Mech)
-6,990	94	403	1.2D+Ftsu+0.5L (Elev/Mech)
-6,990	-38	403	0.9D+Ftsu (Elev/Mech)
23,459	449	975	1.2D+Ftsu+0.5L (Stairs)
23,459	318	975	0.9D+Ftsu (Stairs)
-23,459	449	975	1.2D+Ftsu+0.5L (Stairs)
-23,459	318	975	0.9D+Ftsu (Stairs)

C.15.3.7 Existing Exterior Shear Wall Design for Combined Gravity and Tsunami Loads

The shear wall chosen at Grid Line D and Grid Line 10 from **Figure C-17** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**.

Figure C-155 to **Figure C-162** shows the interaction diagram for the typical exterior shear walls including the tsunami load combinations.

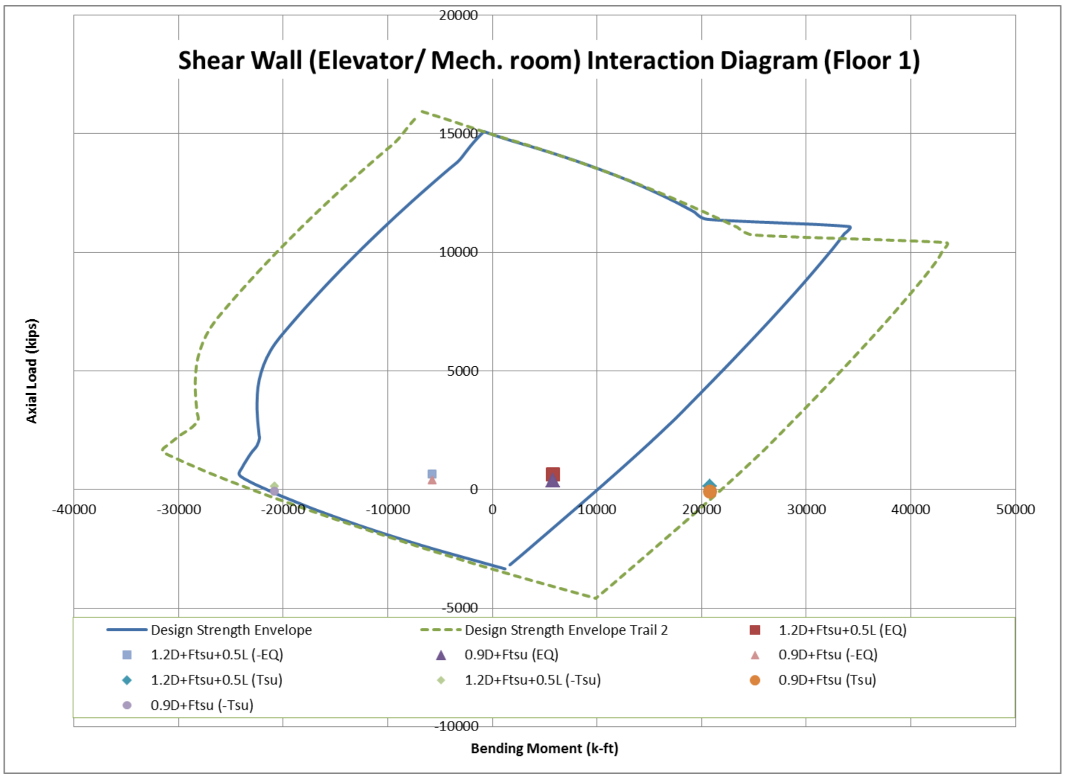


Figure C-155: Interaction diagram for typical ground floor overall elevator shear wall showing tsunami load combinations

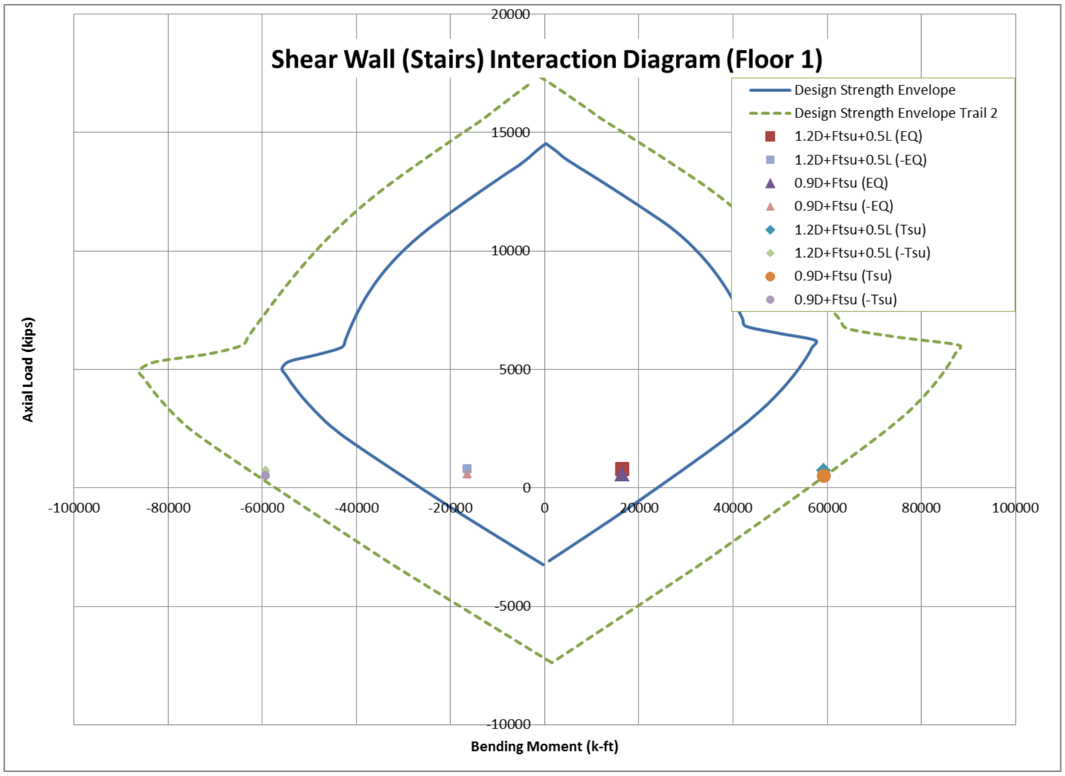


Figure C-156: Interaction diagram for typical ground floor overall stair shear wall showing tsunami load combinations

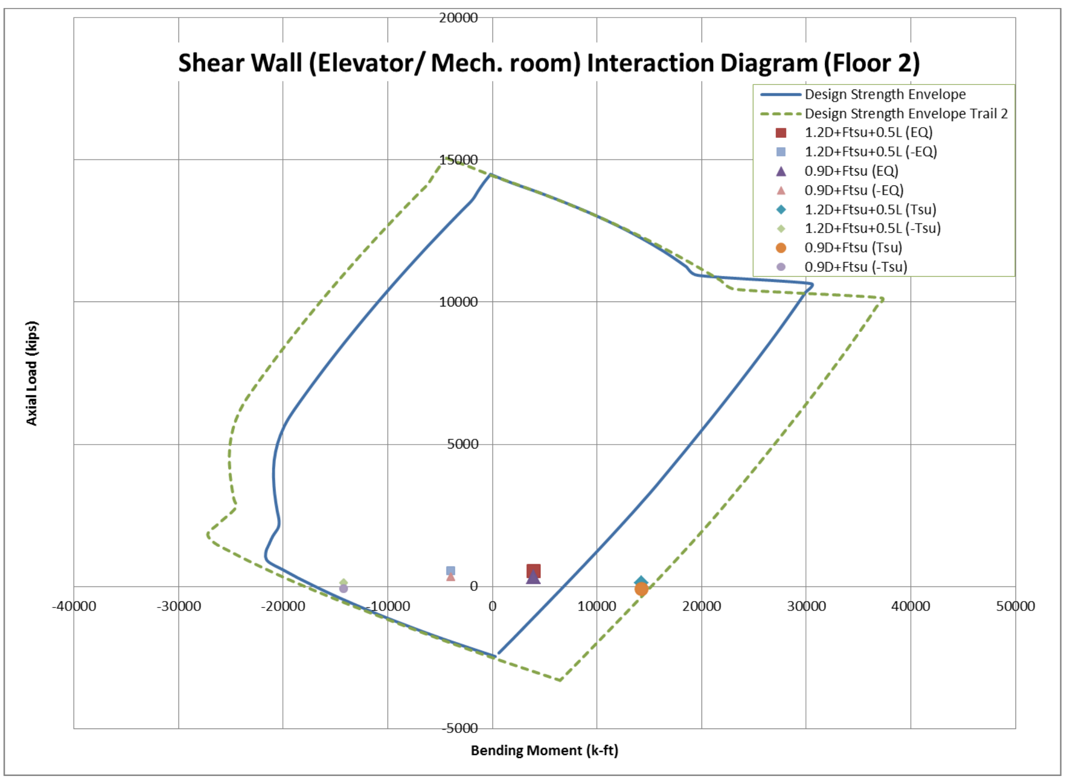


Figure C-157: Interaction diagram for typical 2nd floor overall elevator shear wall showing tsunami load combinations

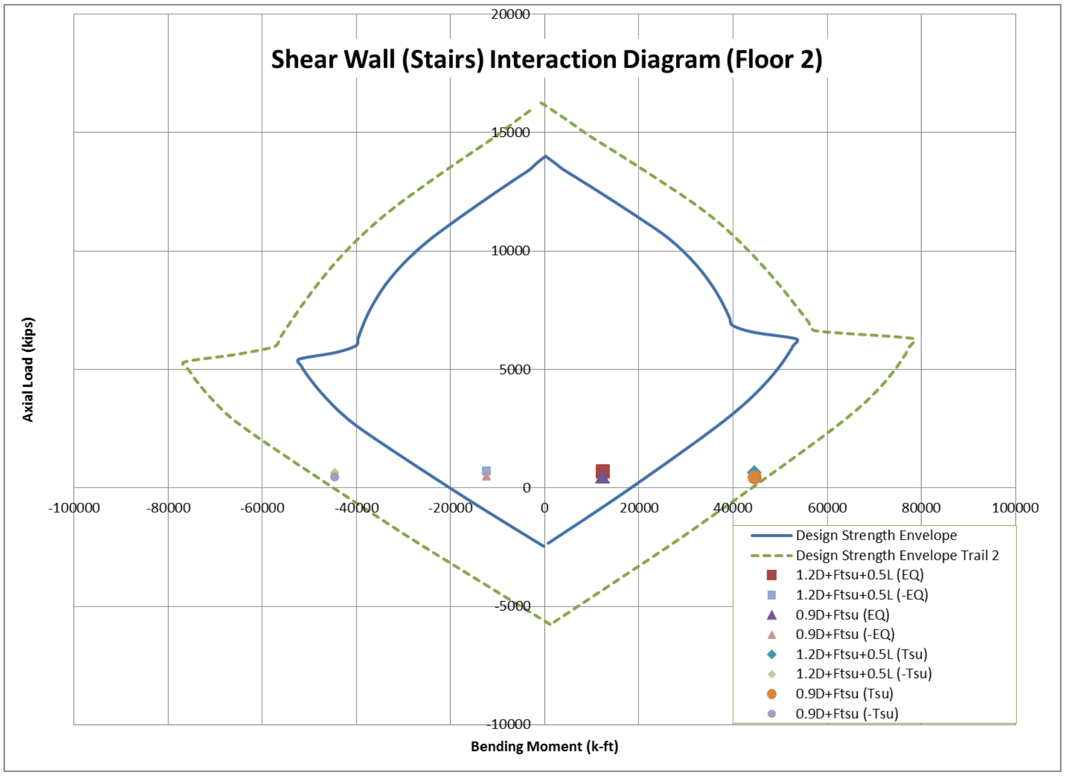


Figure C-158: Interaction diagram for typical 2nd floor overall stair shear wall showing tsunami load combinations

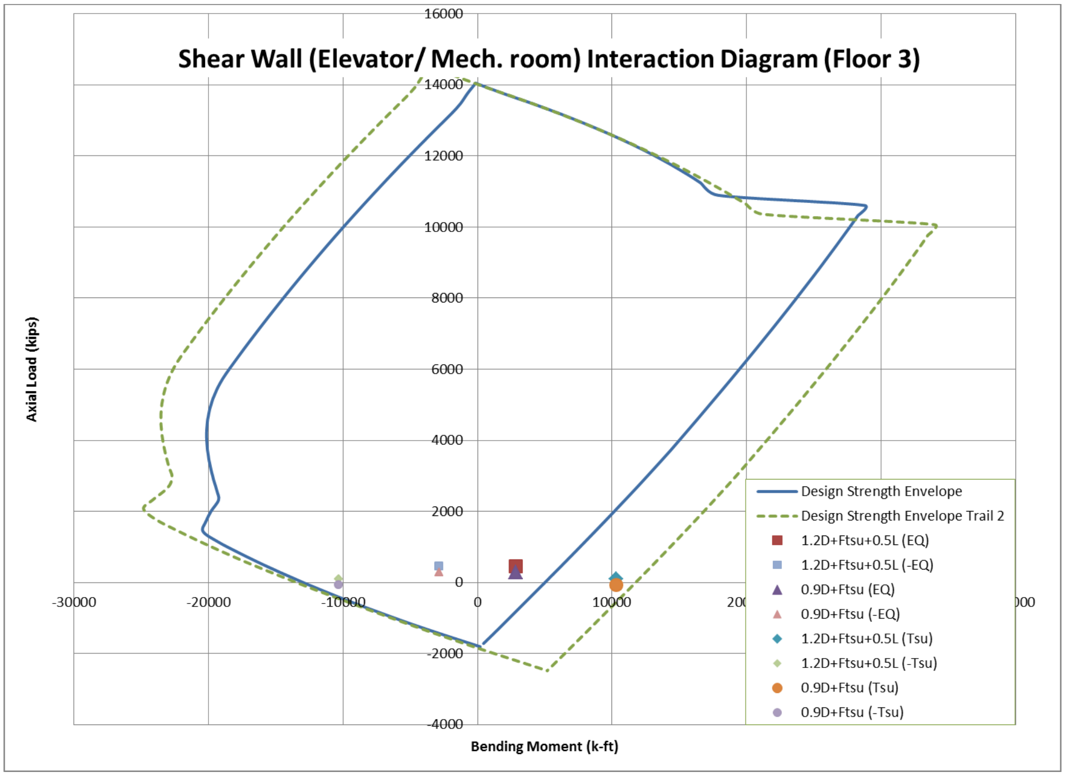


Figure C-159: Interaction diagram for typical 3rd floor overall elevator shear wall showing tsunami load combinations

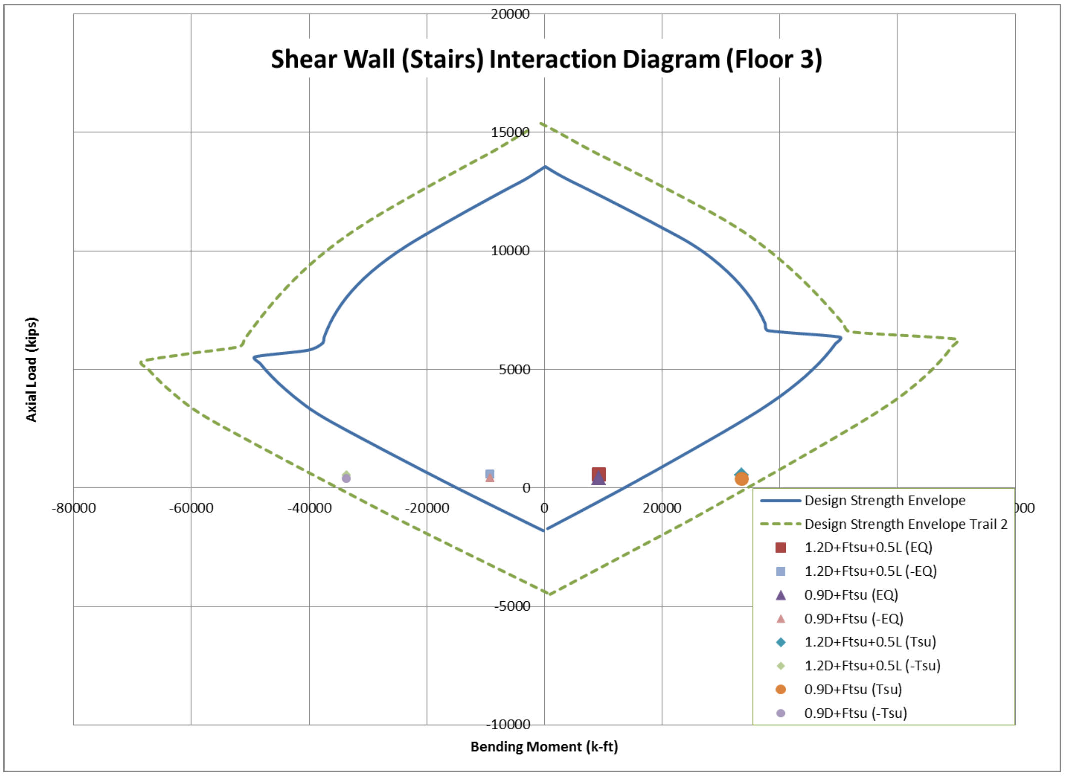


Figure C-160: Interaction diagram for typical 3rd floor overall stair shear wall showing tsunami load combinations

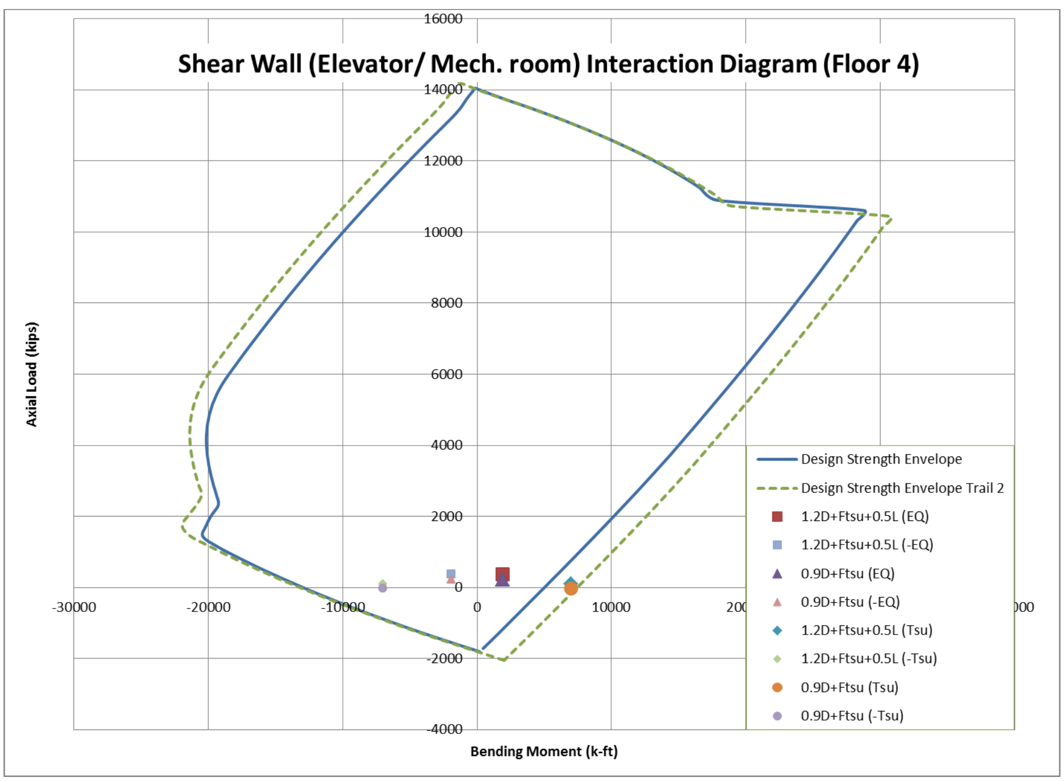


Figure C-161: Interaction diagram for typical 4th floor overall elevator shear wall showing tsunami load combinations

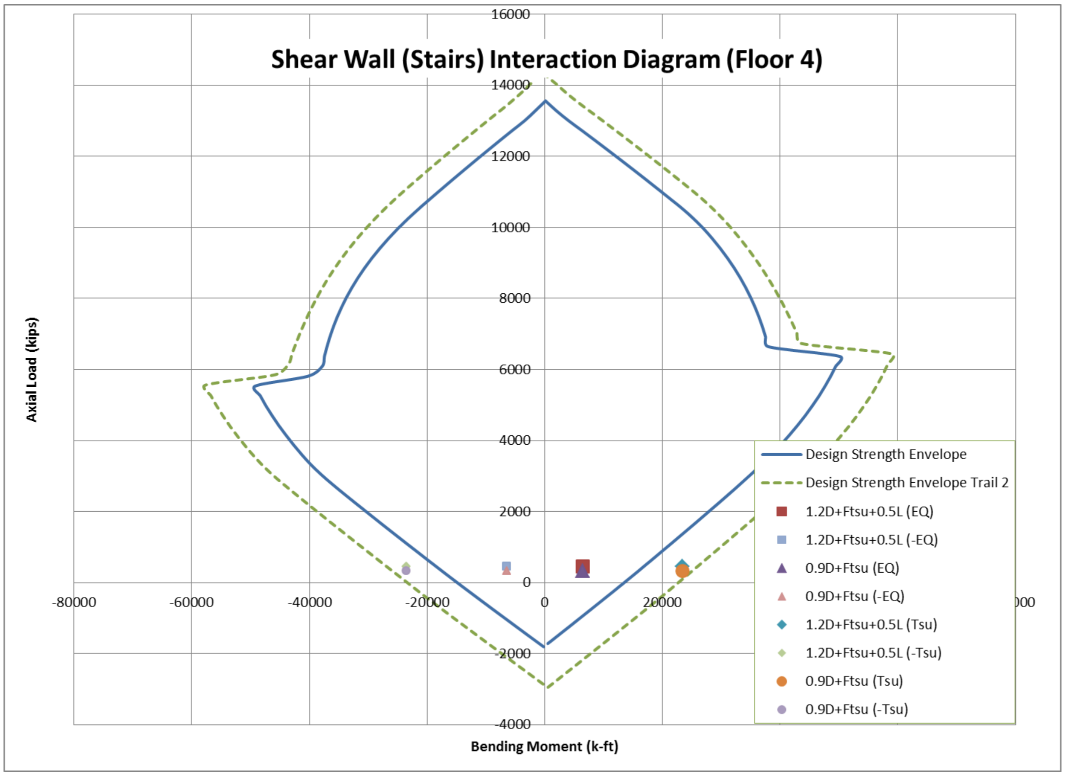


Figure C-162: Interaction diagram for typical 4th floor overall stair shear wall showing tsunami load combinations

C.15.3.8 New Typical Shear Wall Design

The interaction diagrams show that the walls on floors 1 to 4 are inadequate for the bending moments due to hydrodynamic load on the overall shear walls. **Figure C-163** to **Figure C-170** show the revised wall designs required to resist the tsunami loads.

Floor 1 Try 2 (Elevator):

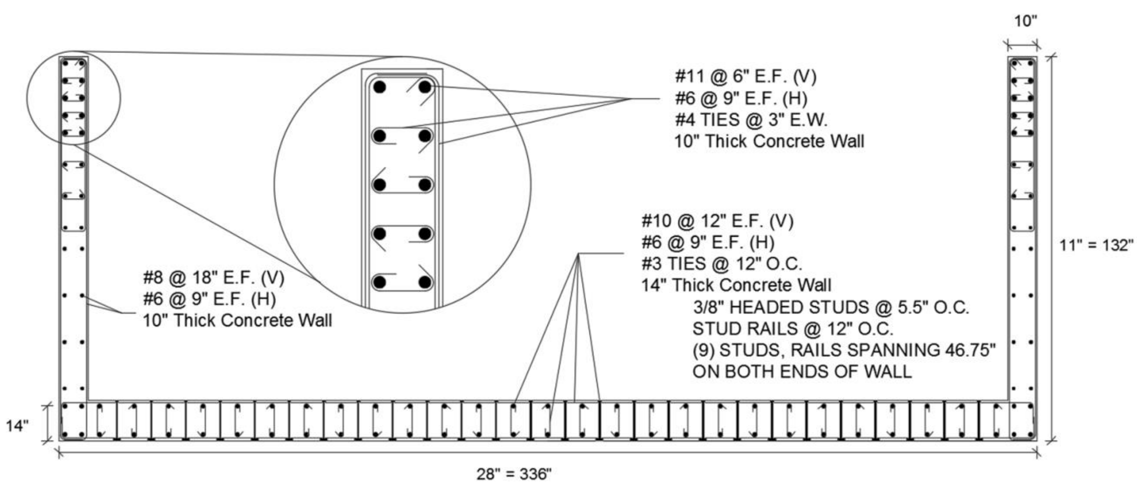


Figure C-163: New Elevator/ Mech. Room shear wall cross-section at the ground floor level

Floor 2 Try 2 (Elevator):

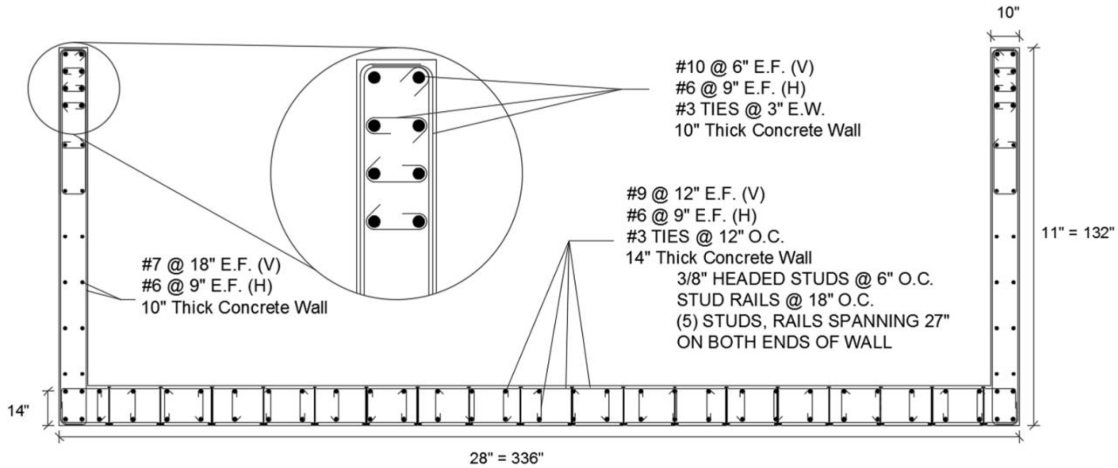


Figure C-164: New overall 2nd floor Elevator/ Mech. Room shear wall

Floor 3 Try 2 (Elevator):

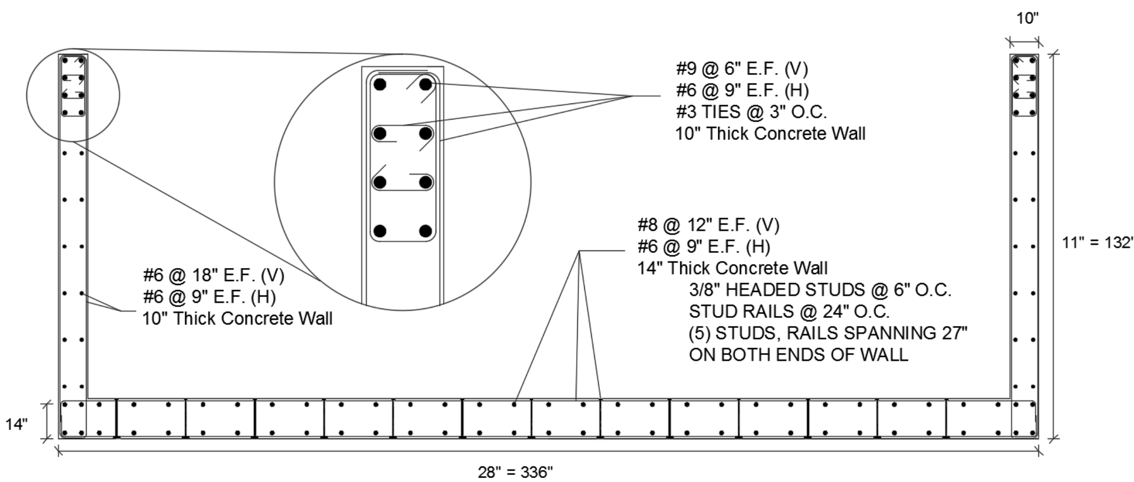


Figure C-165: New overall 3rd floor Elevator/ Mech. Room shear wall

Floor 4 Try 2 (Elevator):

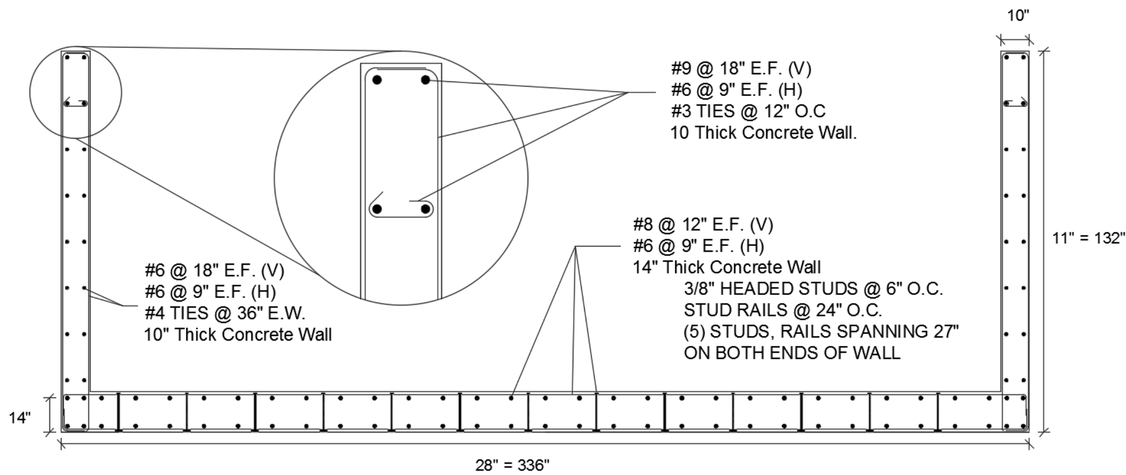


Figure C-166: New overall 4th floor Elevator/ Mech. Room shear wall

Floor 1 Try 2 (Stairs):

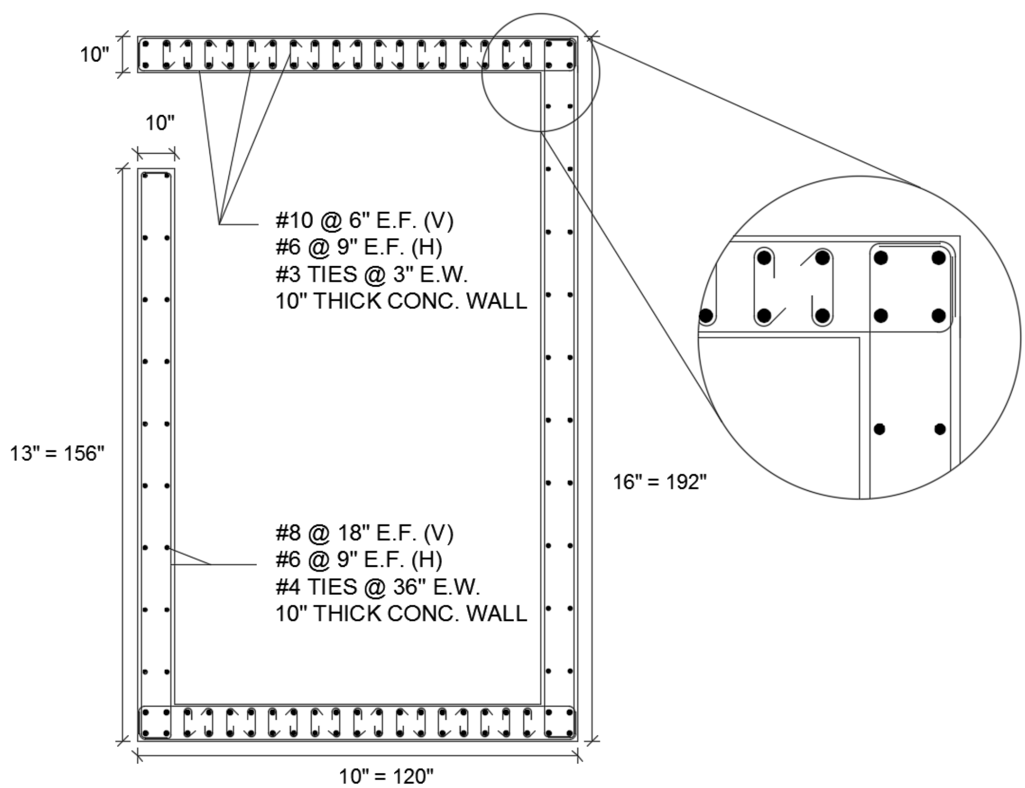


Figure C-167: New stairwell shear wall cross-section at the ground floor level

Floor 2 Try 2 (Stairs):

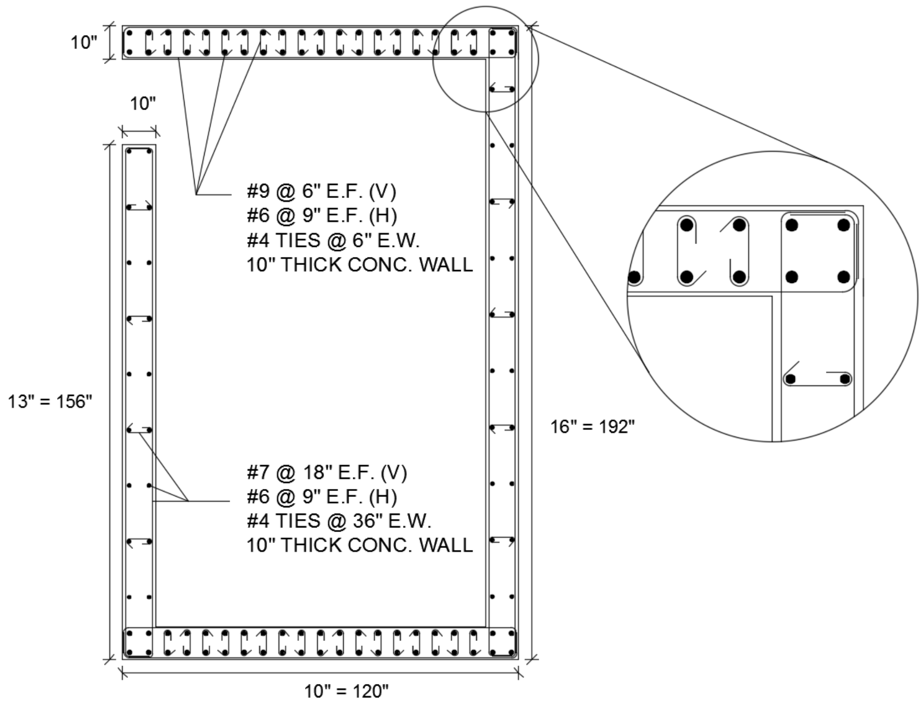


Figure C-168: New stairwell shear wall cross-section at the 2nd floor level

Floor 3 Try 2 (Stairs):

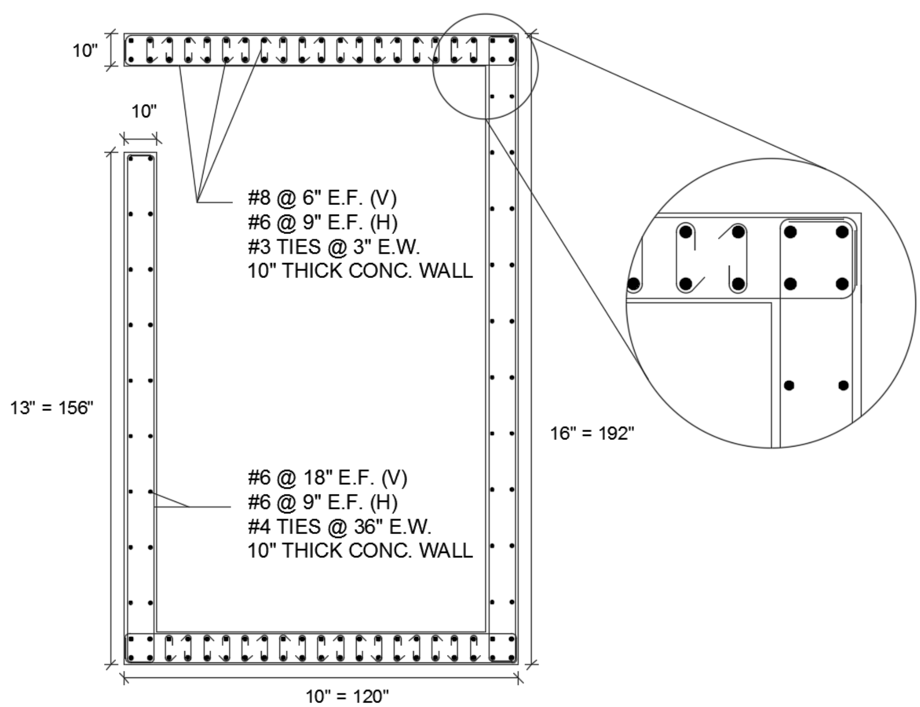


Figure C-169: New stairwell shear wall cross-section at the 3rd floor level

Floor 4 Try 2 (Stairs):

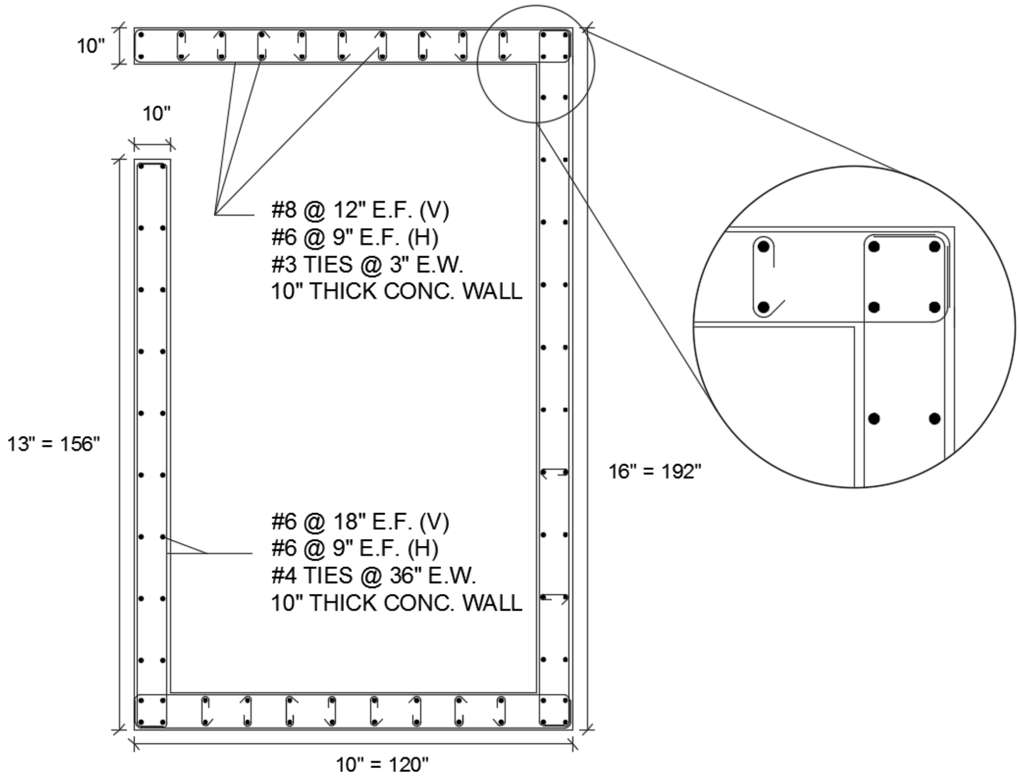


Figure C-170: New stairwell shear wall cross-section at the 4th floor level

C.15.3.9 Exterior Shear Wall Shear Design

Critical Shears:

1st Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$)

Shear Capacity of existing shear wall (Elevator):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\,\lambda\,\sqrt{f'_c}\,h\,d = 2 \times 1\,\sqrt{4000}\, \times 10 \times 105.6/1{,}000$ = 267 kips

$d = 0.8 \times L_w = 0.8 \times 132" = 105.6\ in$

L_w = 11'= 132 in

h = 10" (Thickness)

Ø = 0.75

V_{tsu} = 581 kips > $ØV_c$ = 200 kips ∴ 508 kips needed

$V_S = \frac{A_t f y d}{s} = \frac{(2\times0.44)\times60{,}000\times105.6}{9}$ = 1,239 kips

A_t = 0.44 in (#6 Rebar)

S = 9 in (Spacing)

$ØV_n = Ø(V_C + V_S)$ = 0.75 (267+1,239) = 1,130 kips

At d: V_u = 581 k < ϕV_n = 1,130 k, therefore the wall is adequate for shear.

Shear Capacity of existing shear wall (Stairs):

$ØV_n = Ø(V_C + V_S)$

Where $V_{C1} = 2\,\lambda\,\sqrt{f'_c}\,h\,d = 2 \times 1\,\sqrt{4000} \times 10 \times 154/1{,}000$ = 194 kips

Where $V_{C2} = 2\,\lambda\,\sqrt{f'_c}\,h\,d = 2 \times 1\,\sqrt{4000} \times 10 \times 125/1{,}000$ = 158 kips

$V_C = V_{C1} + V_{C2}$ = 194 + 158 = 352 kips

$d_1 = 0.8 \times L_w = 0.8 \times 192" = 154\ in$

$d_2 = 0.8 \times L_w = 0.8 \times 156" = 125\ in$

L_{w1} = 16'= 192 in

L_{w2} = 13'= 156 in

h = 10" (Thickness)

Ø = 0.75

V_{tsu} = 1,230 kips > $ØV_c$ = 264 kips ∴ 1,288 kips needed

$V_{S1} = \frac{A_t f y d}{s} = \frac{(2\times0.44)\times60{,}000\times154}{9}$ = 901 kips

$V_{S2} = \frac{A_t f y d}{s} = \frac{(2\times0.44)\times60{,}000\times125}{9}$ = 732 kips

$V_s = V_{s1} + V_{s2}$ = 1,633 + 1,225 = 1,633 kips

A_t = 0.44 in (#6 Rebar)

S = 9 in (Spacing)

$ØV_n = Ø(V_C + V_S)$ = 0.75 (352+1,633) = 1,489 kips

At d: V_u =1,230 k < ϕV_n = 1,489 k, therefore the wall is adequate for shear.

By inspection the remaining floors are adequate to resist the tsunami shear force.

D. Waikīkī Design Example – Appendix D

D.1 Project Site

The Waikīkī design example considers a multi-story reinforced concrete building in Waikīkī, Hawaii, at the location shown in **Figure D-1**. The center of the building footprint is located at 21.275736 N; -157.82565 W, which is 170 feet from the shoreline. **Figure D-1** also shows the three topographic transects along which the Energy Grade Line Analysis needs to be applied. The center transect, C, is drawn perpendicular to the shoreline, represented by the average coastline for 500 feet either side of the center transect. The clockwise, CW, and counterclockwise, CCW, transects are generated by rotating the center transect through 22.5 degrees in each direction, about the geometric center of the building plan at the grade plane (ASCE 7 Section 6.8.6.1). Each transect is then extended till it reaches the runup points on the ASCE 7 Tsunami Design Zone map. If the end of a transect falls between two of the runup points, then the runup elevations can be interpolated. The resulting runup elevations for each transect are given in **Figure D-1** along with the approximate inundation limit distances obtained using Google Earth. These inundation limit distances will be revised once the runup elevations are plotted on the respective topographic profiles.

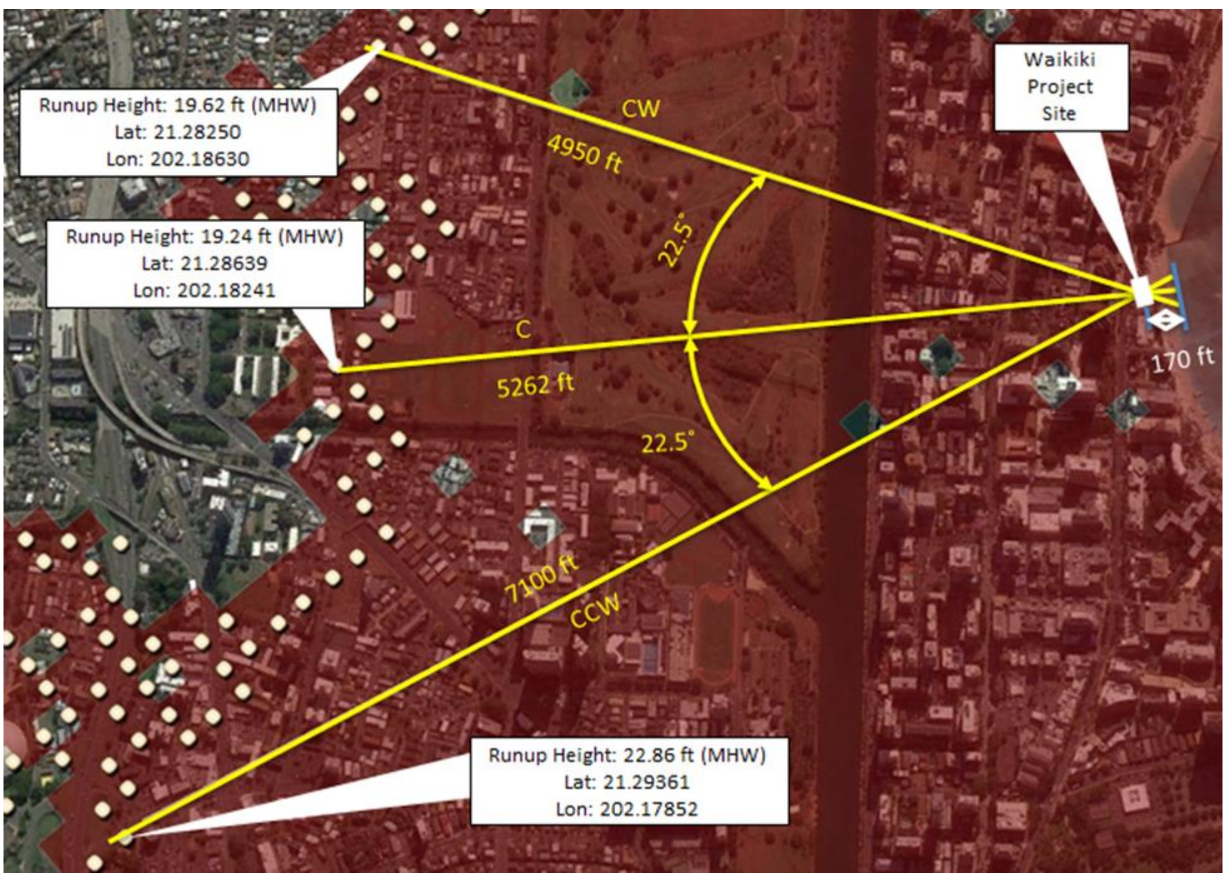


Figure D-1: Location of project site in Waikīkī, Hawaii, relative to inundation line defined by ASCE7-16 Tsunami Design Zone Map. The 22.5° variation in principal flow direction required by Section 6.8.6.1 results in Clockwise (CW) and Counterclockwise (CCW) transects on either side of the Center (C) transect.

Table D-1: Runup elevation and inundation limits for three transects through the Waikīkī Project site.

Transect	Runup Elevation (ft)		Inundation Limit (ft)	
	MHW Reference		From Google Earth	From WGS 84 Transect
	From TDZ	Incl. Sea Level Rise		
Center	**19.42**	**19.69**	**6065**	**5262**
Counterclockwise	**22.86**	**23.13**	**7790**	**7100**
Clockwise	**19.62**	**19.89**	**6000**	**4950**

D.2 Sea Level Change – Section 6.5.3

ASCE 7 Section 6.5.3 requires that any anticipated sea level rise be included in the runup elevation used in the tsunami design. For this example, we will assume sea level change based on a 50 year project life cycle. ASCE 7 Commentary Section C6.5.3 provides a link to

http://tidesandcurrents.noaa.gov/sltrends for historical sea level trends relative to mean sea level (MSL).

From the referenced website the following information is obtained:

"*Honolulu, HI 1612340*

The mean sea level trend is 1.41 mm/year with a 95% confidence interval of +/- 0.21 mm/year based on monthly mean sea level data from 1905 to 2015 which is equivalent to a change of 0.46 feet in 100 years."

The tsunami design should therefore consider the extrapolated prediction of 1.62 mm/year over the 50 year project life cycle. This results in a sea level rise of 81 mm or 3.19" (0. 0.2657 ft). This must be added to the runup elevation for use in the Energy Grade Line Analysis, as shown in **Figure D-1**.

D.3 Topographic Profiles

The topographic profiles along each of these transects was obtained from a Digital Elevation Model, DEM, with the following datums and resolution:

Horizontal Datum: WGS 84

Vertical Datum: MHW

Resolution: 1/3 sec (approximately 10)

The topographic profiles are shown for the Center, Counterclockwise and Clockwise transects in **Figure D-2**, **Figure D-3,** and **Figure D-4** respectively. A horizontal line is plotted on each profile representing the runup elevation (including sea level rise) for each of these transects relative to the MHW datum from **Figure D-1**. The point where this line intersects the profile represents the inundation limit and the starting point for the Energy Grade Line Analysis. The resulting inundation limit should be cross-checked with the Tsunami Design Zone map inundation line to ensure that they are similar distances from the shoreline (See **Figure D-1**). If the TDZ inundation is significantly greater than the first intersection of the runup elevation line with the topographic profile, it may indicate that a region of high ground is present in the inundation zone. The runup elevation must then be modified to match this high ground elevation and the corresponding inundation limit determined where the modified runup elevation next intersects the topographic profile. The resulting values for inundation limit are shown in **Figure D-1** and are used in the EGLA along each transect.

The project site location is also indicated on each plot. For the center transect, the site is located 170 feet from the shoreline (**Figure A-2**). The elevations at the project site vary slightly for the three transects, which can be attributed to slight differences in the elevation data points used to generate each transect profile.

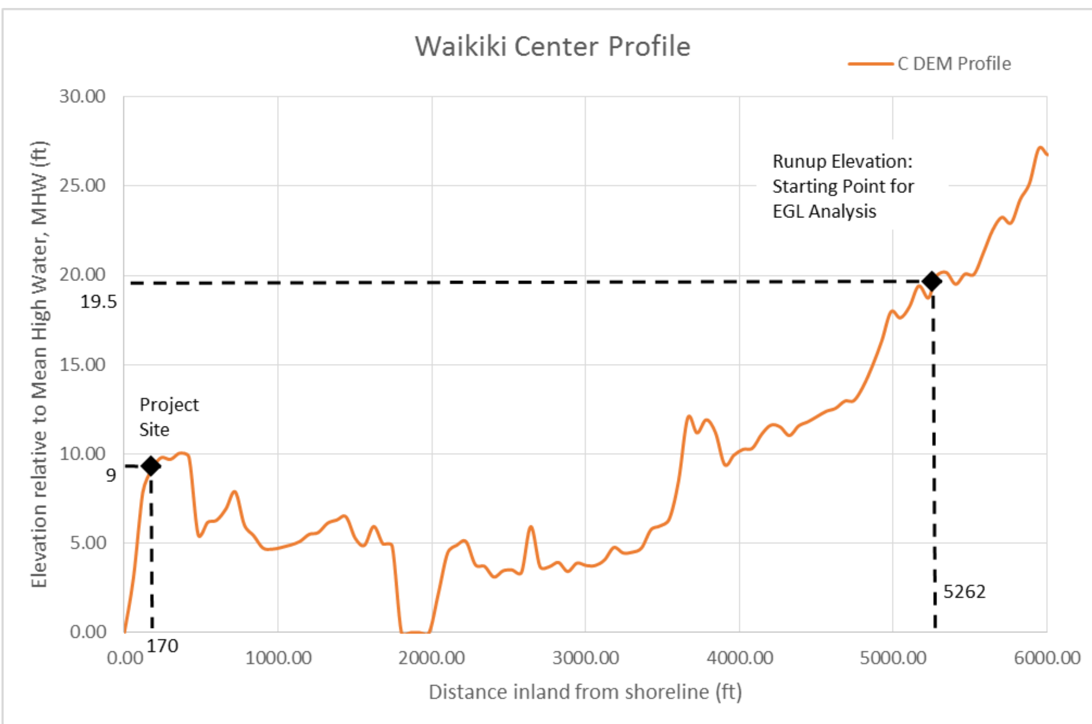


Figure D-2: Topographic profile for Center transect

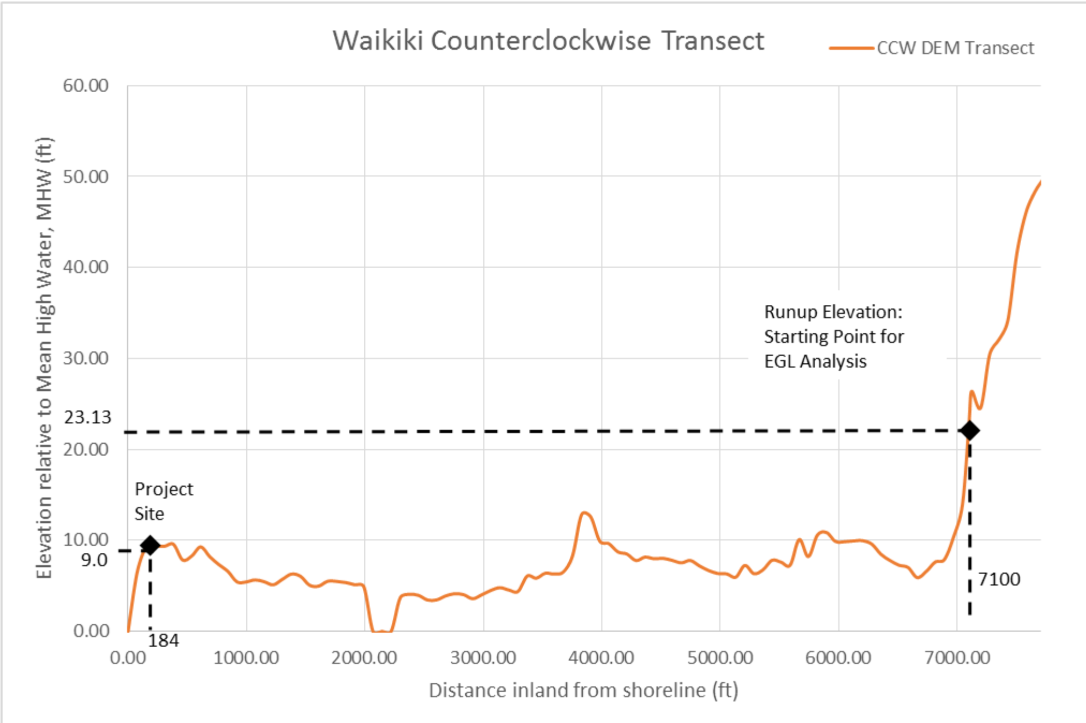


Figure D-3: Topographic profile for Counterclockwise transect

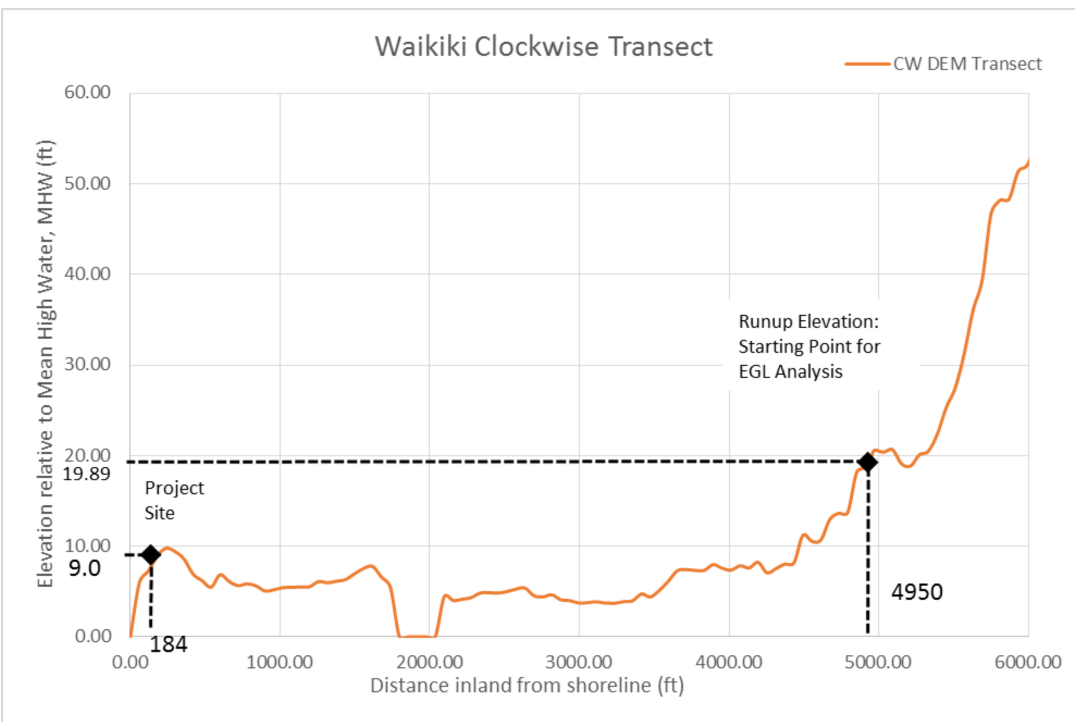


Figure D-4: Topographic profile for Clockwise transect

D.4 Tsunami Bore Determination

In order to determine whether or not a tsunami bore must be considered at the project site, the conditions in ASCE 7 Section 6.6.4 are evaluated for each transect. Tsunami bores shall be considered where any of the following conditions exist:

20. Prevailing nearshore bathymetric slope is 1/100 or milder – NO (See **Figure D-5** and associated discussion).
21. Shallow fringing reefs or other similar step discontinuities – YES
22. Where historically documented – NO.
23. As described in the Recognized Literature – Does not Apply
24. As determined by a site-specific inundation analysis – not required for TRC II buildings.
    Therefore bore loading must be considered in this design.

Figure D-5 shows the approach to determining the average nearshore bathymetric slope so as to determine whether or not tsunami bores need to be considered per **ASCE 7 Section 6.6.4.** A central line is drawn perpendicular to the shoreline. This line is an extension of the center transect running through the project site. The distance from the shoreline to the 100 meter bathymetric line, indicated by the offshore data points in the ASCE offshore wave maps, is then used to determine the average nearshore bathymetric slope. If any of the transect lines does not intersect the 100 meter bathymetric line, this transect can be ignored for the purpose of determining whether or not there is a bore.

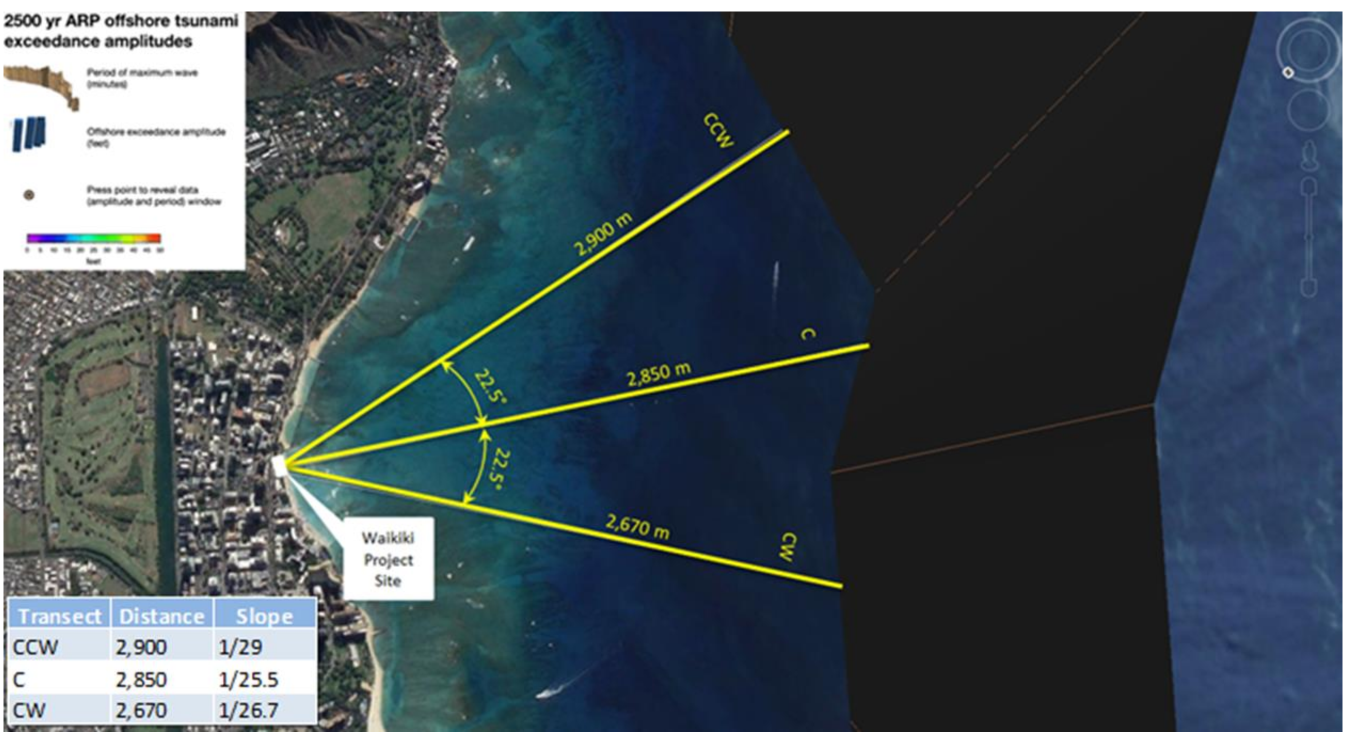


Figure D-5: Determination of average nearshore slope from 100 meter bathymetric line to shoreline along a line perpendicular to the shoreline and lines rotated 22.5 degrees to either side of the center line.

The average nearshore bathymetric slope is then computed using:

$$\emptyset = \frac{100}{distance} \quad \text{in meters}$$

or

$$\emptyset = \frac{328}{distance} \quad \text{in feet .}$$

The table in **Figure D-5** shows that the near shore slope is greater than 1/100, therefore this project site would not create bores through prevailing nearshore bathymetric slope.

D.5 Determination of Inundation Depth and Flow Velocity using EGLA

The Energy Grade Line Analysis (EGLA) is a stepwise procedure starting from the run up elevation at the mapped inundation limit, and working shoreward to get the flow parameters at the site of interest.

A spreadsheet was used to perform this operation along all three transects. The input values were the runup, including sea level rise, referenced to MHW datum (**Figure D-1** column 3), the inundation limit distance determined from the topographic profile (**Figure D-1**column 5), a Manning's coefficient of 0.030 representing "all other cases" from ASCE 7 Table 6.6-1, and α = 1.3 representing bore conditions at the shoreline as specified in ASCE 7 Section 6.6.4. The resulting inundation depth profiles, both with and without the topographical elevation, are shown in **Figure D-6** and **Figure D-7** for the Center transect, **Figure D-8** and **Figure D-9** for the Counterclockwise transect, and **Figure D-10** and **Figure A-11** for the Clockwise transect.

The Counterclockwise transect results in the largest flow depth of 22.88 feet at the project site, which is the value of h_{max} that will be used in the subsequent design calculations.

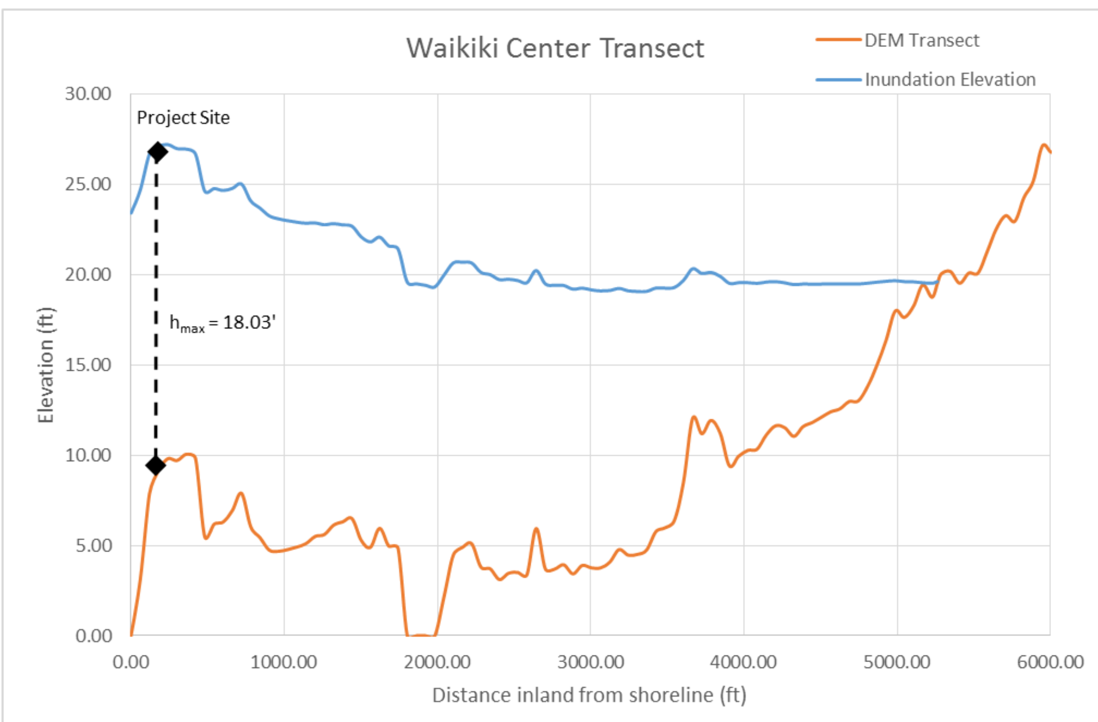


Figure D-6: Inundation depth (*h_i*) over topographic transect from Energy Grade Line analysis for center transect

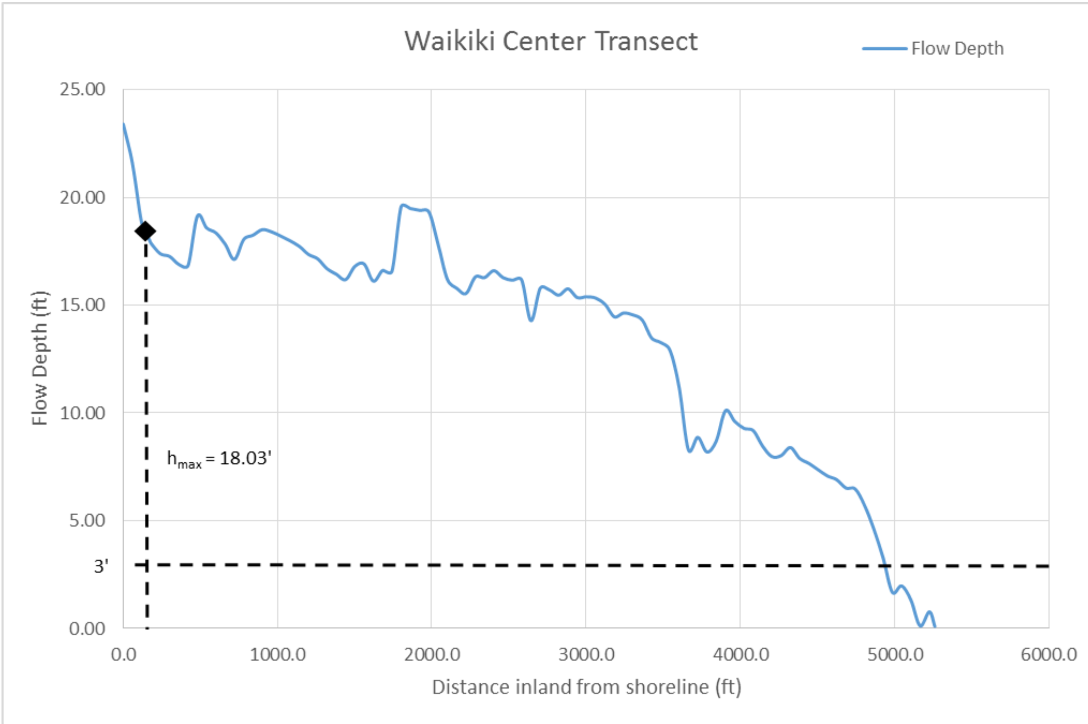


Figure D-7: Inundation depth (h_i) profile from Energy Grade Line analysis for center transect

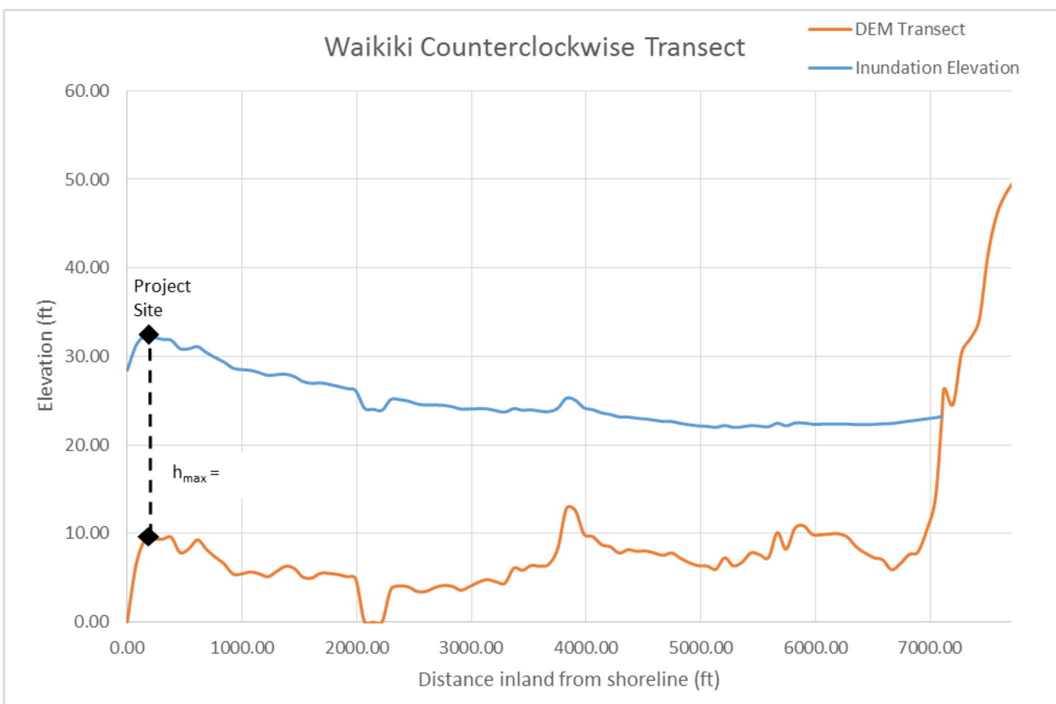


Figure D-8: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for counterclockwise transect

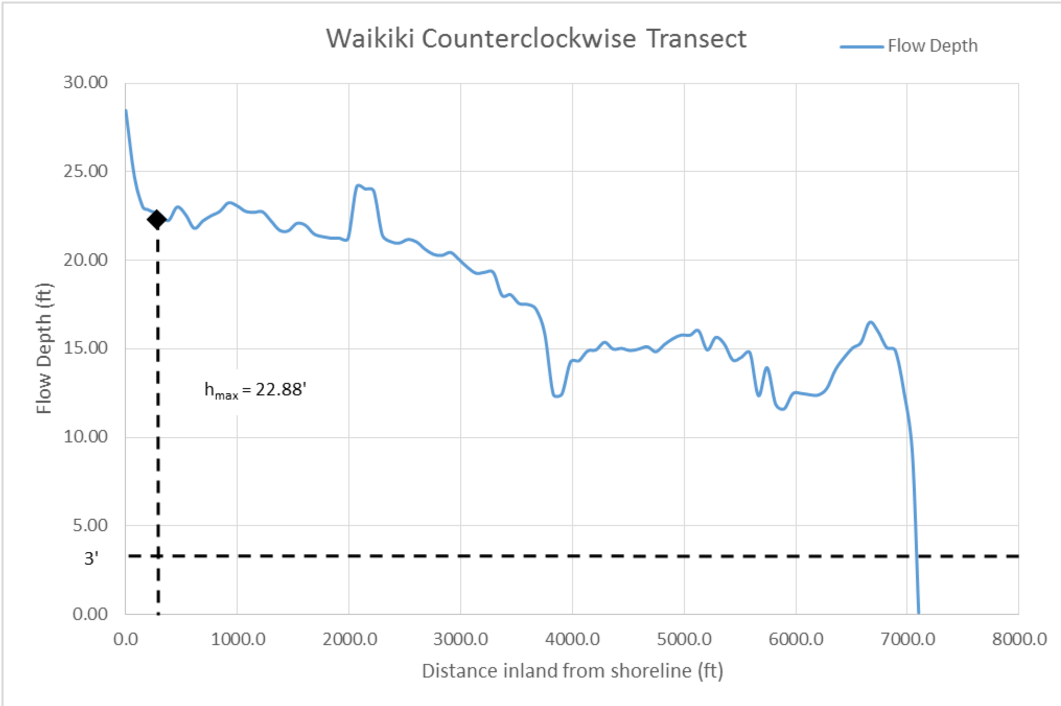


Figure D-9: Inundation depth (h_i) profile from Energy Grade Line analysis for counterclockwise transect

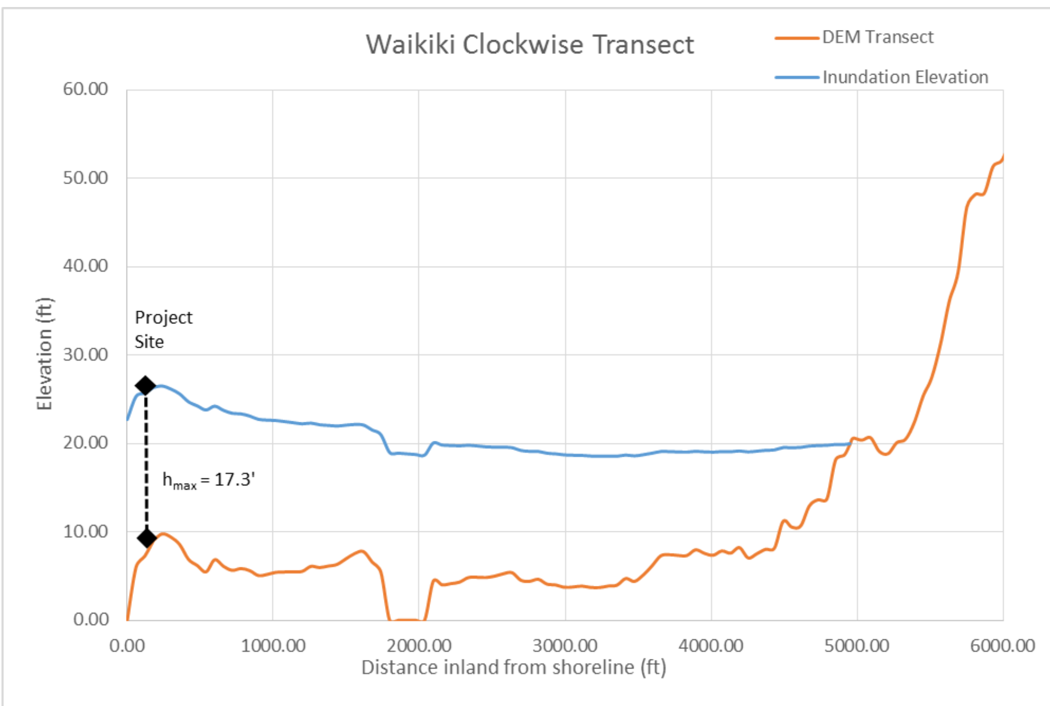


Figure D-10: Inundation depth (h_i) over topographic transect from Energy Grade Line analysis for clockwise transect

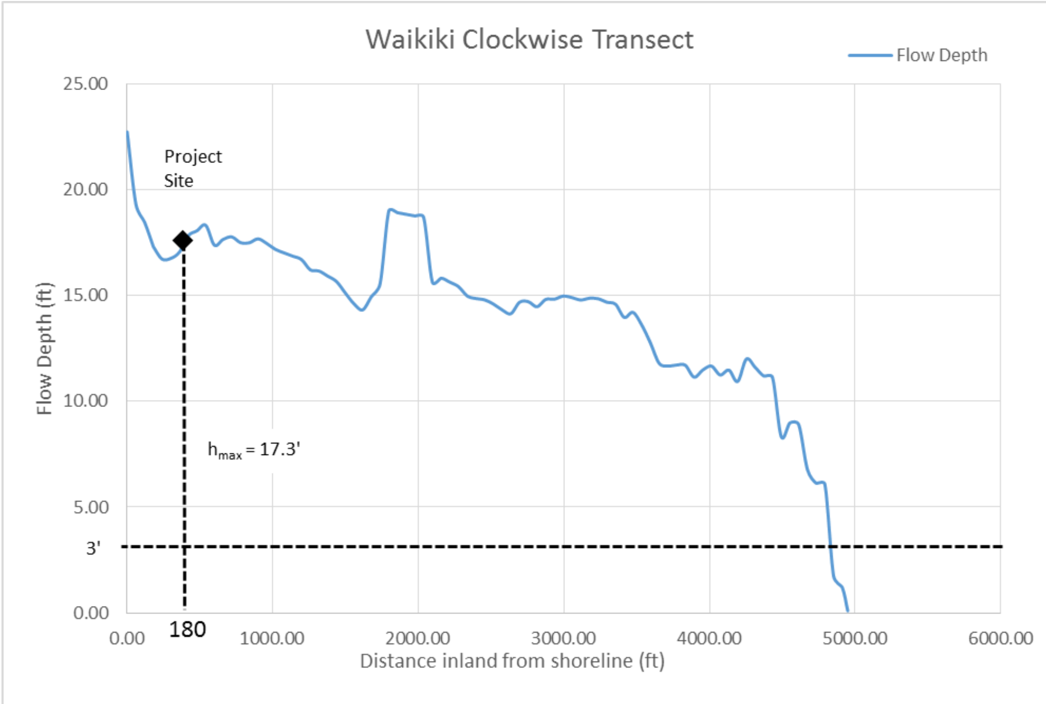


Figure D-11: Inundation depth (h_i) profile from Energy Grade Line analysis for clockwise transect

The flow velocity profiles across each transect as determined from the EGLA are shown in **Figure D-12**, **Figure D-13**, and **Figure D-14** for the Center, Counterclockwise and Clockwise transects, respectively. The minimum flow velocity that may be considered is 10 ft/sec, which is indicated on each of the plots. As with the flow depth, the Counterclockwise transect produces the largest estimate of flow velocity at 34.71 ft/sec, which is the value of u_{max} that will be used in the design calculations.

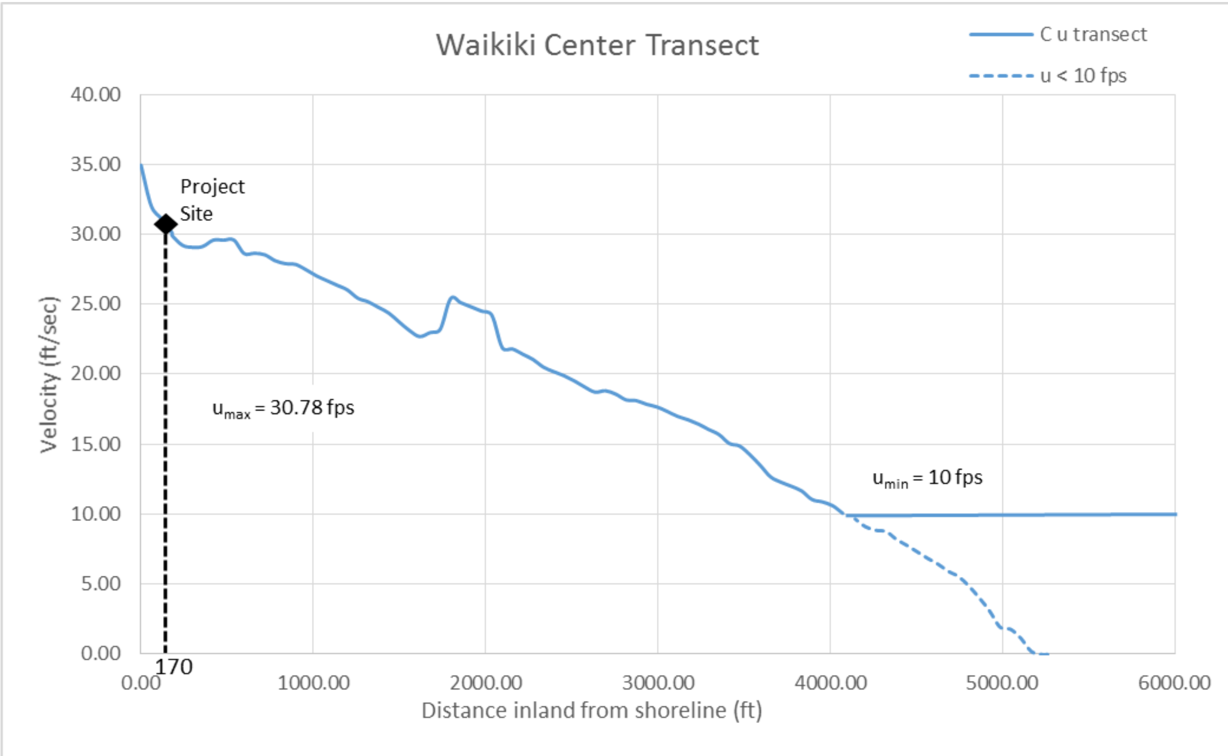


Figure D-12: Flow velocity (u_i) profile from Energy Grade Line analysis for Center transect

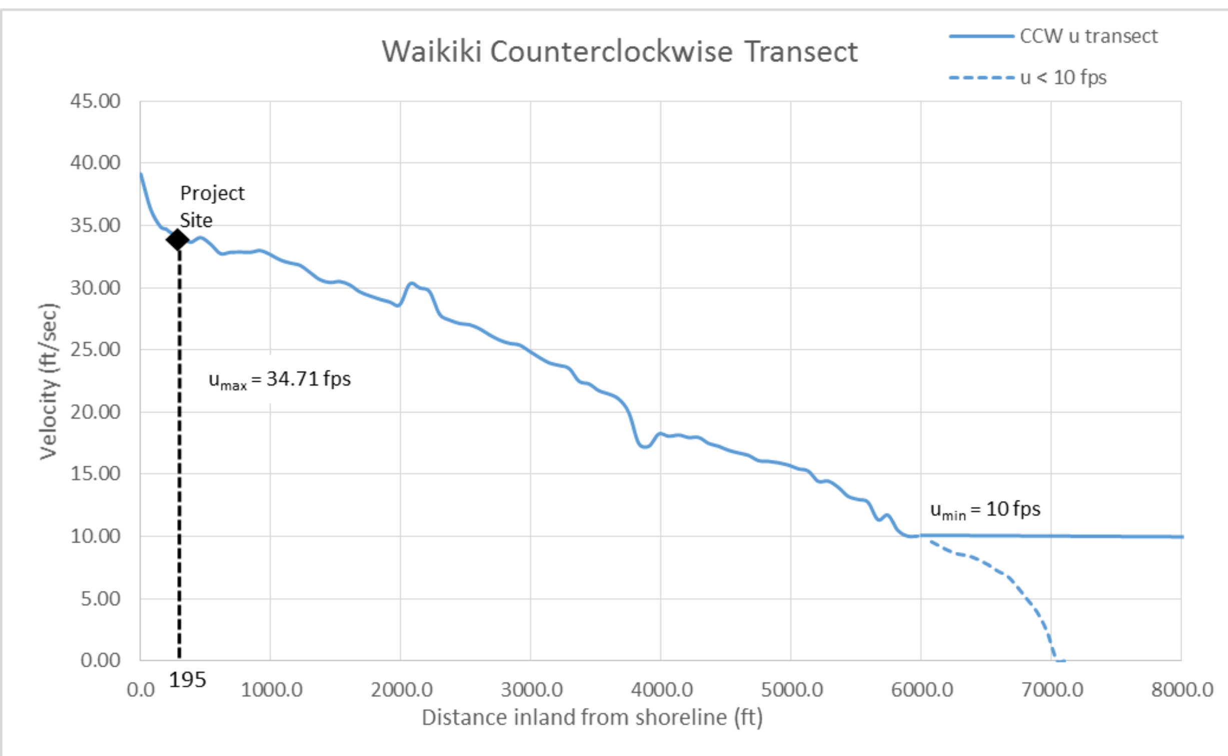


Figure D-13: Flow velocity (u_i) profile from Energy Grade Line analysis for Counterclockwise transect

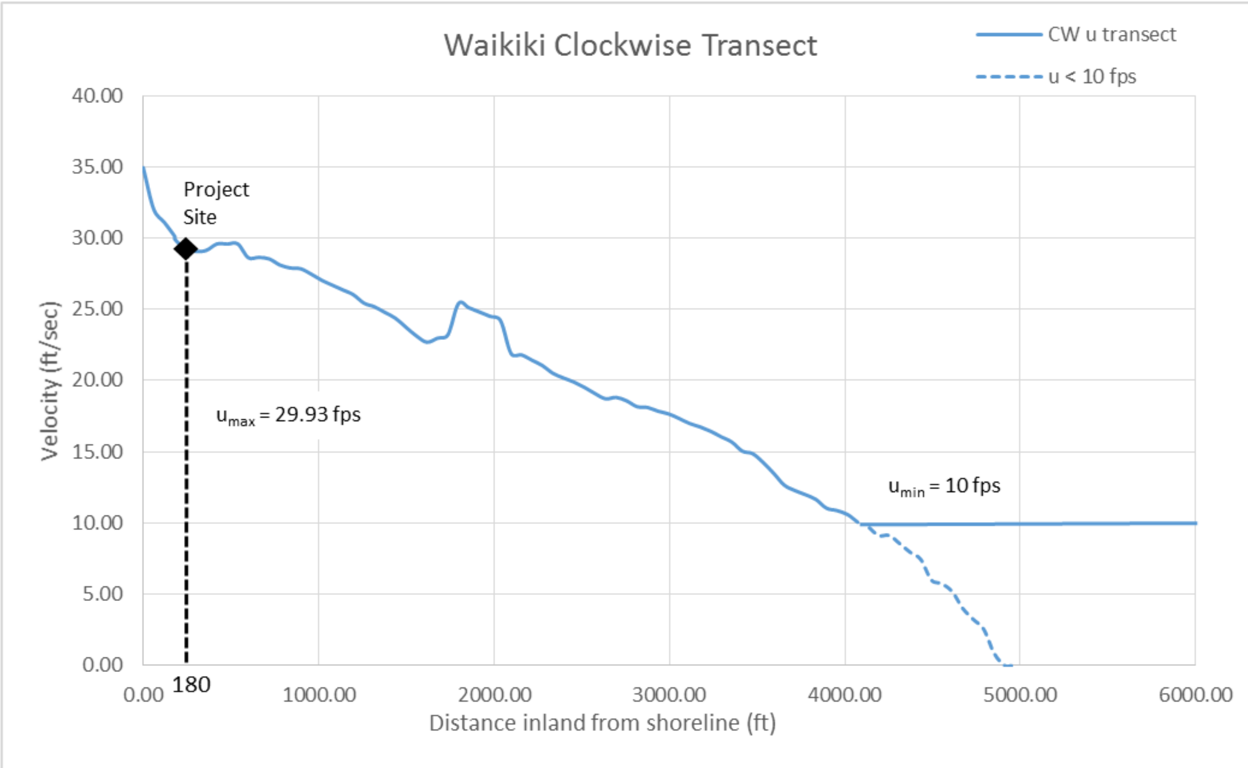


Figure D-14: Flow velocity (u_i) profile from Energy Grade Line analysis for Clockwise transect

All of the flow depths and flow velocities determined from the EGLA are listed in **Table A-2**

Table D-2: Results of Energy Grade Line Analysis for three transects through Monterey project site.

Transect	Maximum Flow Depth, h_{max} (ft)	Maximum Flow Velocity, u_{max} (ft/sec)
Center	18.03	30.78
Counterclockwise	**22.88**	**34.71**
Clockwise	17.3	29.93

D.6 Prototype Concrete Buildings

D.6.1 6-Story Office Building

The 6-story office building consists of a Intermediate Moment Resisting Frame on the perimeter and selected interior frames, and interior gravity columns supporting posttensioned floor slabs (See **Figure D-15)** The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: C

Response Spectrum Parameters: S_s = 0.579, S_1 = 0.17, S_{DS} = 0.516, S_{D1} = 0.24

Structural System Response Factors: R = 5, Ω_o = 3, C_d = 4.5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 74 ft. With a maximum flow depth of 22.88 ft, this building could function as a “Refuge of Last Resort” at the 3rd level (26 ft) up to the roof (if accessible).

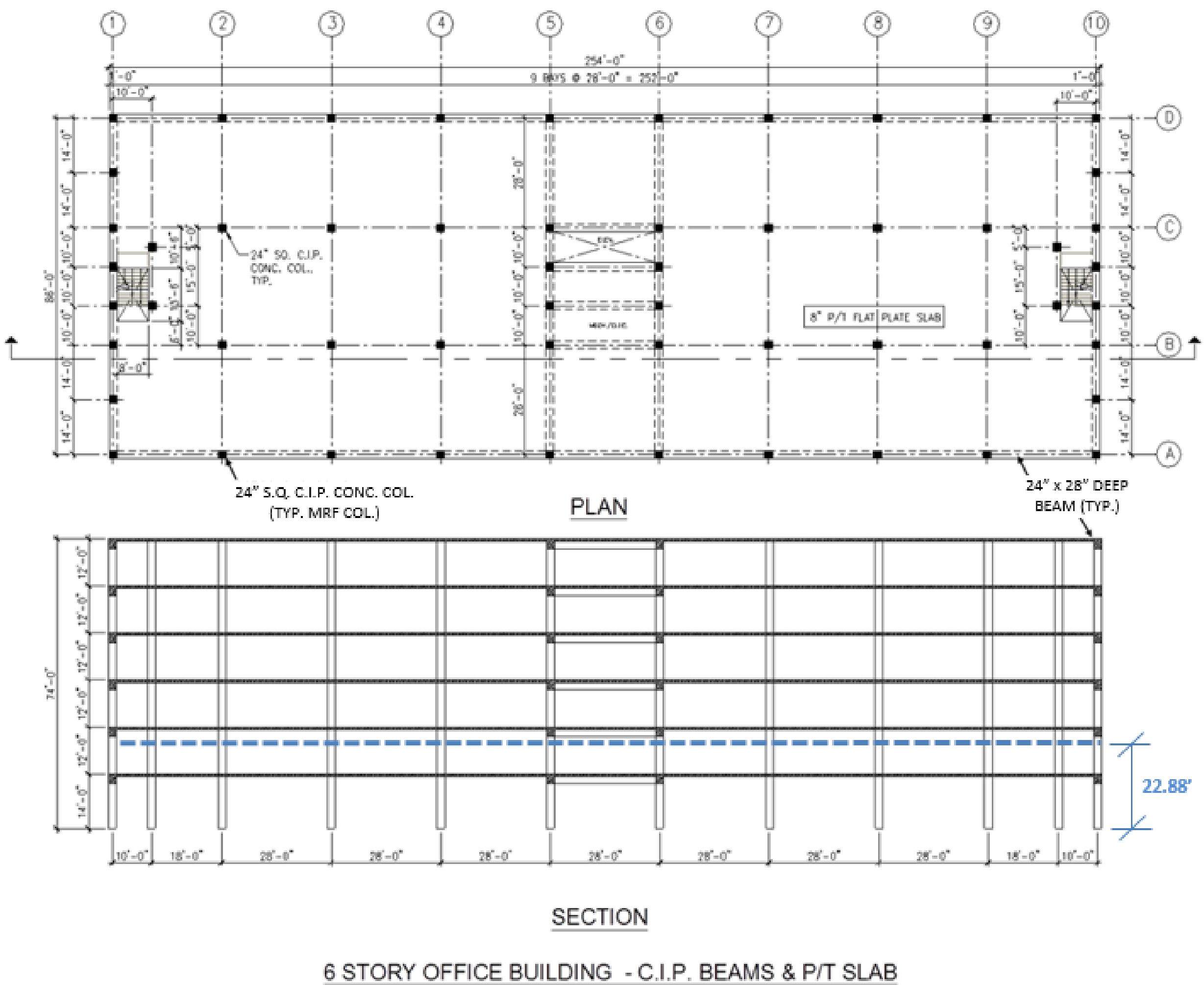


Figure D-15: 6 Story Office Building using Intermediate Reinforced Concrete Moment Frame and posttensioned flat slab supported on gravity columns

D.6.2 7-Story Residential Building

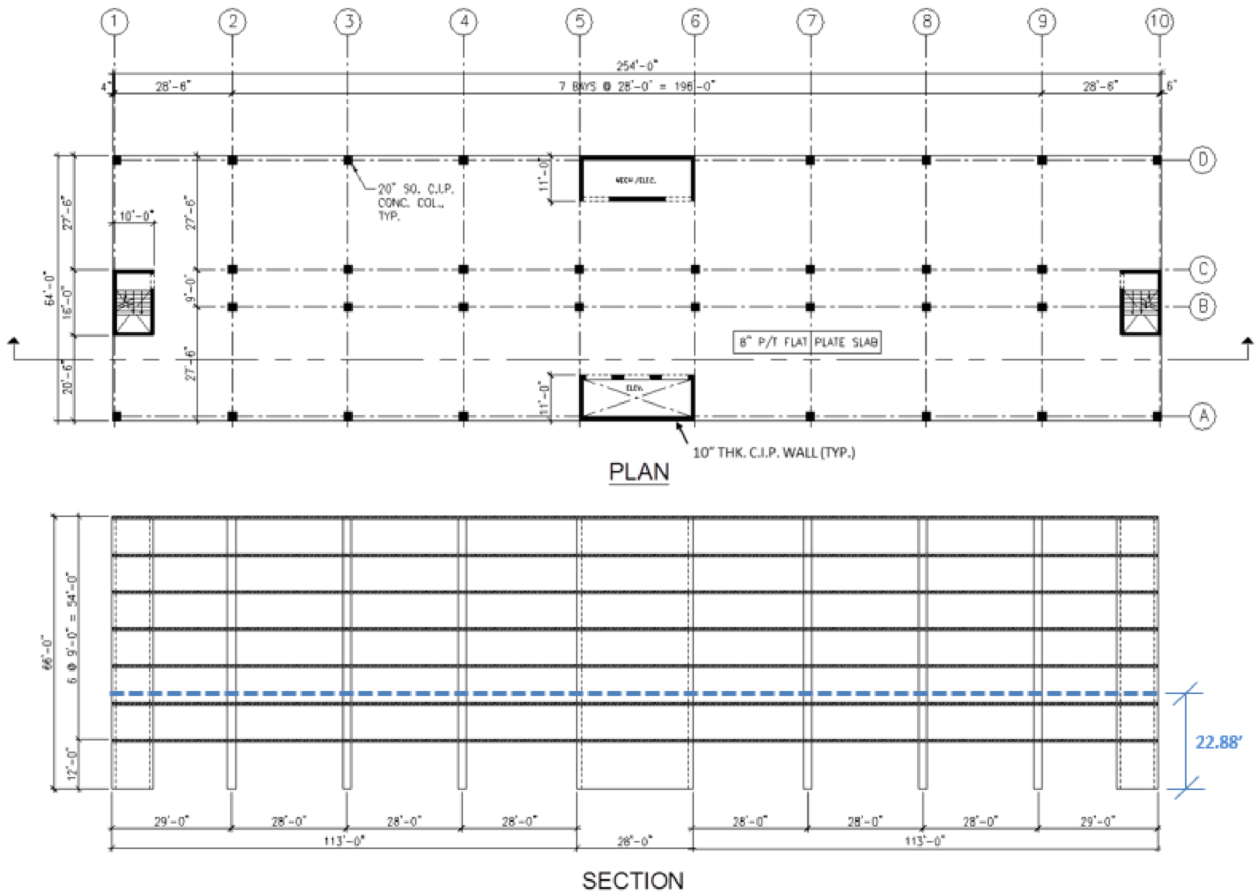
The 7-story residential building consists of a Building Frame System with special reinforced concrete shear walls at exit stairs and elevator core, and interior gravity columns supporting posttensioned floor slabs (See **Figure D-16**). The lateral framing system has been designed for a wind speed of 110 mph and the following seismic design criteria:

Seismic site class: C

Response Spectrum Parameters: S_s = 0.579, S_1 = 0.17, S_{DS} = 1.009, S_{D1} = 0.24

Structural System Response Factors: R = 5, Ω_o = 2.5, C_d = 4.5

This is a Tsunami Risk Category II building with a mean roof height above grade plane of 66 ft. With a maximum flow depth of 22.88 ft, this building could function as a "Refuge of Last Resort" at the 4th level (30 ft) up to the roof (if accessible).



6 STORY RESIDENTIAL BUILDING - SHEARWALLS & P/T C.I.P. SLAB

Figure D-16: 7 Story Residential Building using Special Reinforced Concrete Shear Walls and posttensioned flat slab supported on gravity columns

D.7 Tsunami Loading Summary

Table D-3 gives a summary of the tsunami loads determined for the two prototype buildings located at the selected site. The subsequent sections show the derivation of each of these values.

This example shows detailed calculation of the tsunami loads, along with evaluation of the structural system and components for these loads. Note that these calculations are far more detailed than would be necessary for a typical design project because the intent here is to provide a complete explanation of the various calculations and their application.

Table D-3: Summary of Tsunami Loading for Office and Residential Buildings

Flow Parameters	Office Building	Residential Building
Max. Inundation Depth, h_{max} (ft)	22.88	22.88
Max. Flow Velocity, u_{max} (fps)	34.71	34.71
Overall Building Lateral Loading (kips)		
Load Case 1	2,512	2,512
Load Case 2	4,697	4697
Load Case 3	749	749
Component Loading (kips)		
Exterior Column Hydrodynamic Drag	793 [2]	793 [2]
Interior Column Hydrodynamic Drag	80.9	67.4
Exterior Column Debris Impact	107.25 [3]	107.25 [3]
Exterior Wall Debris Impact	-	107.25 [3]
Wall and Slab Loading (psf)		
Hydrodynamic Pressure on Walls	-	2,651
Stagnation Pressure in Mech/Elec Rm	-	1,325 [5]
Surge Uplift on Elevated Slabs	-	20

[1] Including effect of debris damming, C_{cx}, applied to column tributary width.

[2] Limited by log crushing capacity.

[3] Stagnation pressure acting outwards on structural walls and floor slab enclosing Mech/Elec room corresponding to the maximum velocity and corresponding flow depth.

D.8 Assumed Conditions

The following conditions are assumed to apply for this example:

25. The building is oriented with the longitudinal axis parallel to the shoreline.
26. The building has no basement.
27. The foundation system consists of deep piles with pile caps supporting all shear walls and all exterior columns.
28. The ground floor slab-on-grade has isolation joints at all columns, structural walls and grade beams.

29. The top of the first floor windows is 8 feet above grade, with the window sill at 3 ft.
30. The building location is not in the vicinity of a shipping container storage yard or port facility, and is therefore not subject to debris impact from shipping containers, ships or barges.
31. The non-structural exterior cladding spans vertically between floors.

D.9 Tsunami Design for Office Building

D.9.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

In lieu of performing detailed hydrostatic and hydrodynamic analysis, **Eqn. 6.10.1-1** provides a simplified but conservative estimate of the maximum lateral load on the building.

$$\mathrm{f_{uw}} = 2.5\mathrm{I_{tsu}}\gamma_\mathrm{s}\mathrm{h^2_{max}} = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 22.88^2 = 92.14\ \mathrm{kip/ft}$$

Assuming $\mathrm{C_{cx}} = 0.7$ controls per **Section 6.8.7**

Then $\quad \mathrm{F} = 0.7 \times 254 \times 92.14 = 16{,}382\ \mathrm{kips}$

This lateral load can be compared with 0.75 Ω_o E_h = 0.75 x 3 x 1,565 = 3,521 kips < 16,382 kips. The detailed analysis for LC2 and LC3 should therefore be performed as shown above, in which case the LFRS is adequate with no strengthening.

D.9.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

D.9.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} $\le h_{max}$ = 22.88ft

$\le$ 14 ft (ground floor story height)

$\le$ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

For the sake of illustration, if we had assumed *that the ground floor consists of structural grade beams and integral slab on grade without isolation joints, and that the soil allowed ground water pressure increase below the building (ie. sandy or gravely subsoil), the buoyancy would need to be considered as follows:*

Section 6.9.1, Eqn. 6.9-1 $F_v = \gamma_s V_w$ = (1.1x64.0)(254’ x 88’ x 8’)/1000 = 12,588 kips

Apply load combination: 0.9*D* + F_{TSU} + 1.2 H_{TSU}

where H_{TSU} = 0 *since scour is assumed uniform around the building perimeter*.

and building dead weight, D = 16,000 kips, *including foundation*.

Therefore net uplift = - 0.9 x 16,000 + 12,588 = -1812 kips, downward.

Overall uplift would therefore not be a concern, even if the ground floor were a structural slab capable of resisting the associated buoyancy pressures. This example also ignores any uplift resistance provided by the deep foundations.

In combination with buoyancy, Load Case 1 requires application of the associated hydrodynamic drag on the entire building.

Section 6.10.2, Eqn. 6.10-.2 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254’ overall width of building

h = 8’

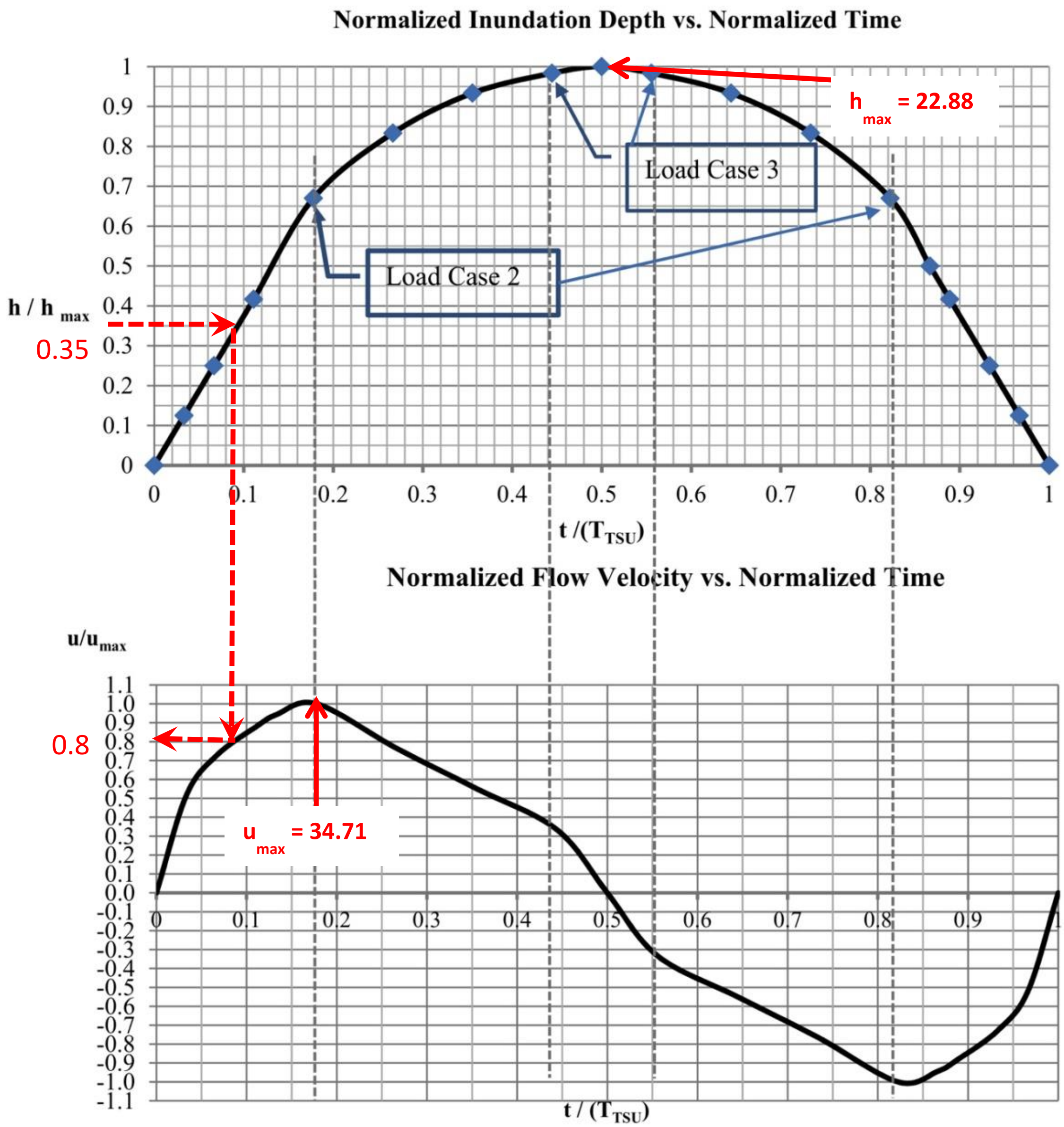


Figure D-17: Determining "u" for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/22.88 = 0.35. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.09. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.8. Therefore the flow velocity is u = 0.8 x 34.71 = 27.77 fps.

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 27.77^2)/1000 = 2{,}512\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure at the first floor level would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail so as to relieve lateral load on the structural frame. Note that only portion of this load will go to the second floor slab, which therefore has to be resisted by the lateral force resisting system. The majority of the load will go directly to the grade beam/foundation system. The entire lateral load must be resisted by the deep foundation assuming maximum scour has already occurred.

D.9.2.2 Load Case 2: Maximum Flow Velocity

In this particular example, LC1 and LC2 are very similar for the overall building, but the following calculation is shown for completeness.

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is 2/3h_{max} = 2/3 x 22.88 = 15.25 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.306 (**Table 6.10-1** based on B/h_{sx} = 254'/15.25' = 16.65)

Since the inundation depth of 20.93 feet exceeds the bottom of the second floor beams (14' – 24"/12) = 12', the inundated area of the beams must be included in the closure coefficient, which is given by:

h_{sx} = 15.25 ft.

$h_{col\ EQ}$ = 15.25'-28"= 12.92' (Clear height of submerged Moment Resisting Frame columns)

$A_{col\ EQ}$= 12.92'x2'x40 = 1,033 ft^2 (40 MRF earthquake columns each 2' wide)

$h_{col\ Grv}$= 15.25'-8"=14.58' (clear height of submerged gravity load columns)

$A_{col\ Grv}$= 14.58'x2'x16 = 466 ft^2 (16 gravity load column, each 2' wide)

A_{wall} = 0 ft^2 (no walls in MRF structure)

A_{beam} = 28"x254'x1= 593 ft^2 (1x28" deep beam goes above 2nd level beam)

$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum((1033+466)+0)+1.5\times 593}{254'\times 15.25'} = 0.617 < 0.7$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

$u = u_{max}$ = 34.71 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2}\times 2.2 \times 1.0 \times 1.4 \times 0.7 \times 254(15.25 \times 34.71^2)/1000 =$ $4{,}697\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal or inland elevation of the building over a height of 15.25 feet above grade as shown in **Figure D-18**. The lateral force resisting system for the structure at the first and second floor levels would be evaluated for this load.

D.9.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 22.88 ft. and the flow velocity is $1/3u_{max}$ = 1/3 x 22.88 = 11.57 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.25 (**Table 6.10-1** based on B/h_{sx} = 254/22.88 = 11.1)

Since the inundation depth of 22.88 feet exceeds the bottom of the second floor beams (14' – 28") = 11.67', the inundated area of the beams must be included in the second floor closure coefficient, which is given by:

h_{sx} = 22.88 ft.

$h_{col\ EQ}$ = 22.88'-28"= 20.55'

$A_{col\ EQ}$= 20.55'x2'x40 = 1,644 ft^2

$h_{col\ Grv}$= 22.88'-8"= 22.22'

$A_{col\ Grv}$= 22.21'x2'x16 = 711 ft^2

A_{wall} = 0 ft^2

A_{beam} = 28"x254'x1= 592 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum((1644+711)+0)+1.5\times 592}{254\prime \times 22.88\prime} = 0.558 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 11.57 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(22.88 \times 11.57^2)/1000 = 749\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 22.88 feet above grade as shown in **Figure D-19**. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth for hydrodynamic load and debris impact.

D.9.3 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category C, **Section 6.8.3.4** does not apply and the lateral force resisting system (LFRS) must be evaluated for the full tsunami loads. For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 4,697 kips applied over a height of 15.25 ft. Portion of this load will be resisted by the grade beam/foundation system as shown in **Figure D-18**, resulting in a tsunami base shear of 2,541 kips at the ground floor level.

This tsunami base shear must be distributed up the height of the building as shown in **Figure D-18**. Similarly, for Load Case 3, the tsunami loads are distributed as shown in **Figure D-19**. The ETABS model used for the original wind and seismic analysis of the building was used for this analysis, resulting in the column forces shown in **Figure D-20** for the first floor and **Figure D-21** for the second floor.

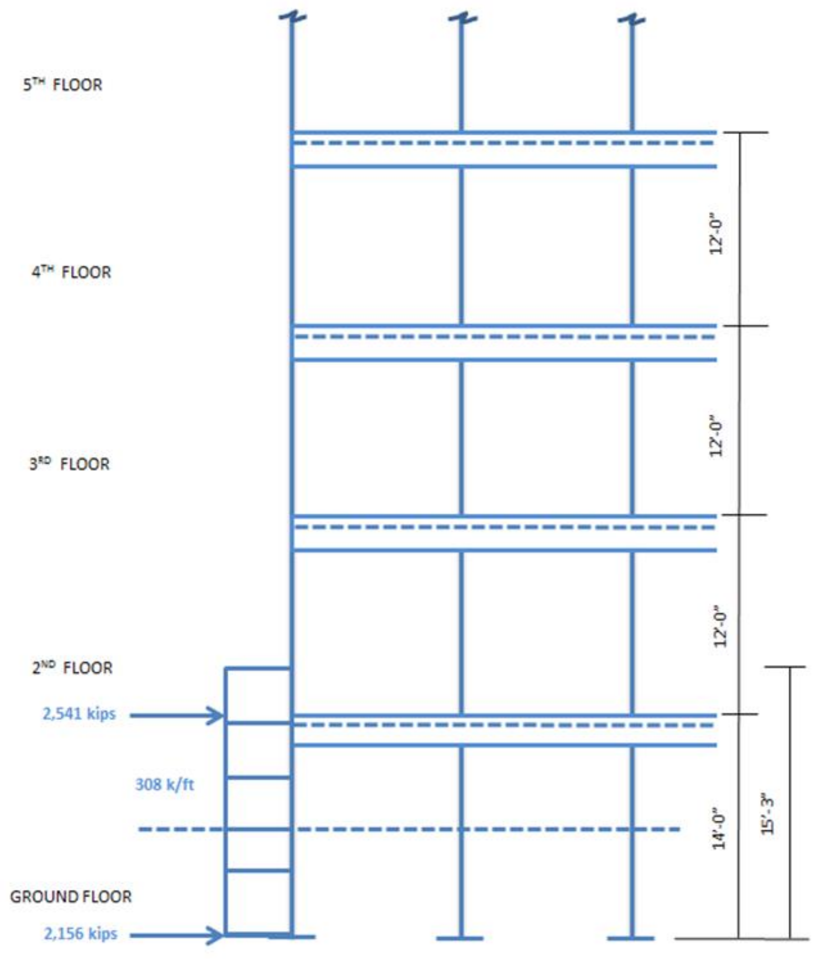


Figure D-18: LC2 Tsunami loads on overall Waikīkī office building

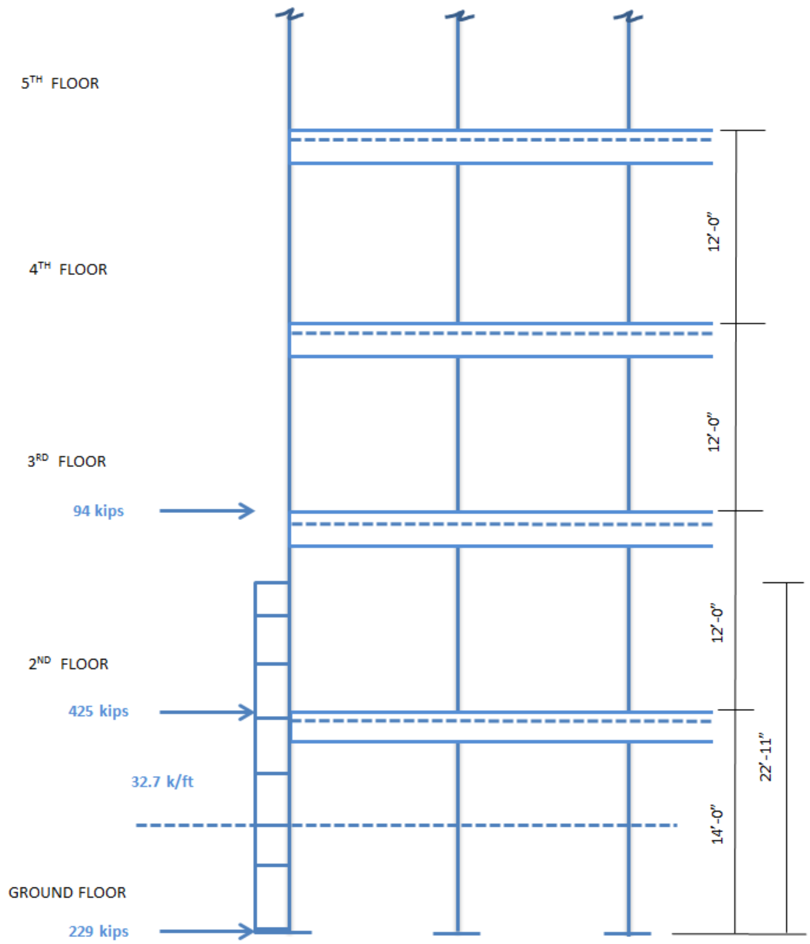


Figure D-19: LC3 Tsunami loads on overall Waikīkī office building

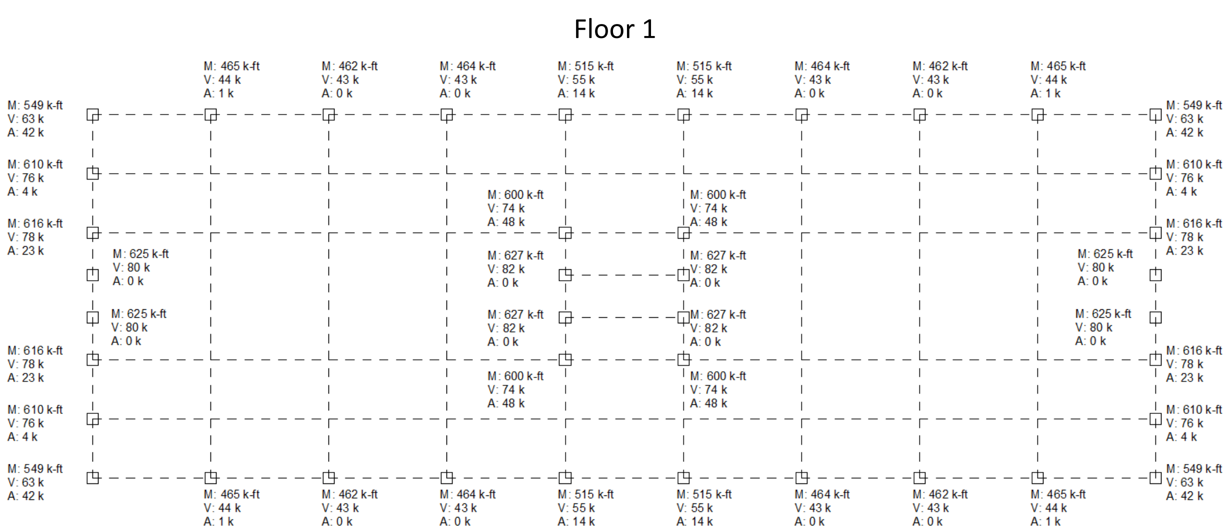


Figure D-20: Maximum forces in the first floor columns due to tsunami base shear

Floor 2

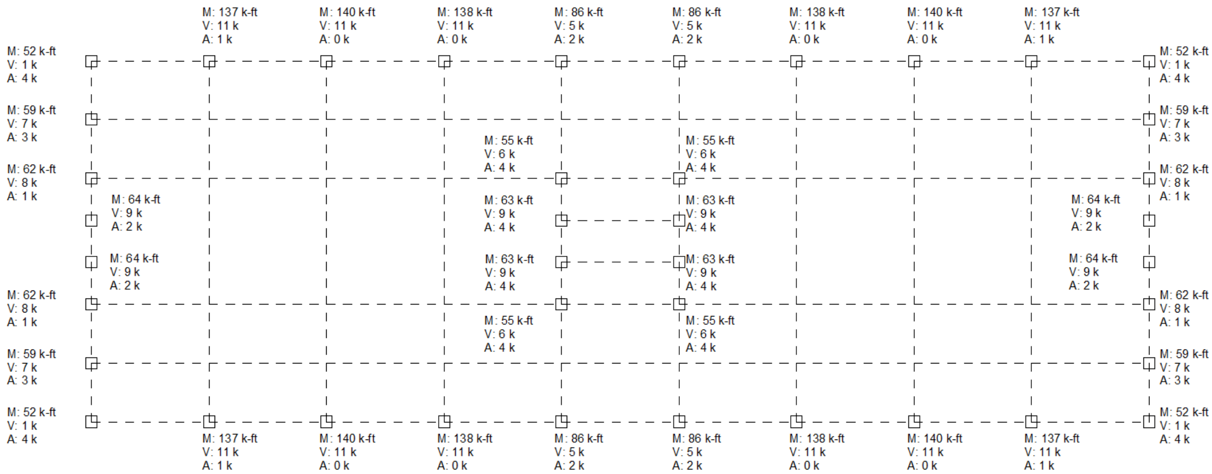


Figure D-21: Maximum forces in the second floor columns due to tsunami base shear

D.10 Component Design

D.10.1 Drag Force on Components - Section 6.10.2.2

D.10.1.1 Exterior Columns

For Load Case 1, the exterior cladding is assumed to remain intact. Since the cladding spans vertically between floors for this example building, none of the hydrodynamic lateral load in LC1 will be applied directly to the ground floor columns. [*Note that if the exterior cladding were supported by girts which transferred lateral load to the columns, then the columns would need to be designed for this load*.]

For Load Cases 2 and 3, the exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** will require that C_d = 2.0 and the width dimension, b, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Previous calculation of C_{cx} showed that the default value of 0.7 controls for LC2 and LC3 for this building, Therefore b = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 15.25 ft and u_{max} = 34.71 fps.

The hydrodynamic drag is computed using **Eqn 6.10-4** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(15.25 \times 34.71^2)/1000 = 793\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 793/15.25= 51.95 kips/ft over the lower 15.25 feet of the column. The column must be designed for this load combined with gravity loads using the load combinations in **Section 6.8.3.3**. In addition, because the exterior columns are part of the LFRS, these component loads must be combined with the systemic forces and the column designed for the combined loads.

D.10.1.2 Interior Columns

Interior columns are 24" (2 ft) square R.C. columns. For Load Case 1, the interior is not yet inundated, so there are no hydrodynamic loads on the interior columns. The controlling load case will be LC2, when the inundation depth is h_e = 15.25 ft and u_{max} = 34.71 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**)

Therefore $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 2.0(15.25 \times 34.71^2)/1000 = 80.9\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 80.9/15.25 = 5.3 kips/ft over the lower 15.25 feet of the column. This load must be combined with gravity loads using the load combinations in **Section 6.8.3.3** and the column capacity verified.

D.10.2 Other Hydrodynamic Loads

No other hydrodynamic load conditions apply to this building since there are no structural walls and the spandrel beam is integral with the slab so the lateral load on the beam will transfers directly to the slab diaphragm.

D.10.3 Debris Impact Loads - Section 6.11

The inundation depth at the site exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads per **Section 6.11**.

D.10.3.1 Detailed Debris Impact Calculation for Office Building

Wood Logs and Poles - Section 6.11.2

The nominal maximum instantaneous debris impact force is given by **Eqn. 6.11-2** as:

$$F_{ni} = u_{max}\sqrt{km_d}$$

Where u_{max} = 34.71 fps

$k = EA/L$ for the wood log with a minimum value of 350 k/in (4.2×10^6 lb/ft)

m_d = 1000/32.2 = 31.1 slugs for the minimum 1000 lb log.

Therefore: $F_{ni} = u_{max}\sqrt{km_d} = 34.71\sqrt{4.2 \times 10^6 \times 31.1}/1000 = 396.43\ kips$

The design instantaneous debris impact force is then given by **Eqn. 6.11-3** as:

$$F_i = I_{tsu}C^{0F_{ni}} = 1.0\ \times 0.65 \times 396.43 = 257.68\ kips$$

The impulse duration is given by **Eqn. 6.11-4** as:

$$t_d = \frac{2m_d u_{max}}{F_{ni}} = \frac{2\times 31.1\times 34.71}{396{,}436} = 0.00544\ sec$$

The column can be designed using a dynamic analysis by applying an impulsive rectangular pulse with magnitude F_i and duration t_d. Alternatively an equivalent elastic static analysis can be performed of the column subjected to F_i multiplied by a dynamic response factor, R_{max}, given in **Table 6.11-1**. The ratio of impact duration to natural period of the impacted structural element is obtained using t_d and the natural period of the column assumed to be fixed-fixed. For this case, the natural period is given by;

$$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}}$$

Where L = unbraced column length = 14' – 28" = 11.67 ft for the ground floor columns.

ρ = column mass per unit length = 2'x2'x150pcf/32.2ft/s^2 = 18.63 slugs/ft

E = modulus of elasticity of the column concrete = 3600 ksi = 518.4x10^6 psf

I = moment of inertia of column section = $bd^3/12$ = 2x2^3/12 = 1.33 ft^4

Therefore $$T_{col} = 2\pi\left[\frac{L^2}{22.373}\right]\sqrt{\frac{\rho}{EI}} = 2\pi\left[\frac{11.67^2}{22.373}\right]\sqrt{\frac{18.63}{518.4\times 10^6\times 1.33}} = 0.00265\ sec$$

The ratio of impact duration to column natural period is therefore t_d/T_{col} = 0.00544/0.00265 = 2.055.

Table 6.11-1 gives the dynamic response factor R_{max} = 1.5, therefore the equivalent static load is given by;

$$F_{es} = R_{max}F_i = 1.5\times 257.68 = 386.52\ kips.$$

This exceeds the maximum required impact force of 107.25 kips, therefore the column can be evaluated for a lateral point load of 107.25 kips applied at locations which are critical for flexure and shear.

D.10.4 Impact by Vehicles – Section 6.11.3

The impact force is given as $F_i = I_{tsu}$ x 30 = 30 kips. This will not control over the log impact load determined above.

D.10.5 Impact by Submerged Tumbling Boulder and Concrete Debris – Section 6.11.4

Because h_{max} = 22.88 ft > 6 ft, an impact force of $F_i = I_{tsu}$ x 8 = 8 kips shall be applied at 2ft above grade. This will not control over the log impact load determined above.

D.10.5.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330C_0 I_{tsu} = 330\times .65\times 1.0 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force will be reducible to 50%, or 107.25 kips. This load must be applied to the exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with hydrodynamic loads on the column, but it must be combined with systemic loads if the member is part of the lateral force resisting system. In the event that this load exceeds the column capacity, the column can be strengthened, or a detailed debris impact dynamic analysis can be performed. Debris impact loads are not applied to interior columns.

D.11 Column Design for Tsunami Loads

D.11.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 **Figure D-15**. The column is part of the lateral force resisting system for longitudinal seismic load designed and detailed for Seismic Design Category C. The column has been designed for gravity and seismic loads resulting in the cross-section shown in **Figure D-22** and **Figure D-23** at all floor levels. The column will now be checked for tsunami load combinations.

Seismic design of the columns requires additional column ties to ensure ductility of the yield zones at each end of the column. These zones have a length equal to the maximum column cross-section dimension, in this case 24 inches. The critical shear force in this yielding zone occurs at a distance "*d*" from the top and bottom of the column, where d = 24 - 1.5 - 0.5 - 0.5 = 21.5 in. The critical shear force for the internal section of the column occurs at "d + h" from the edge of the column, where d + h_c = 21.5 + 24 = 45.5 in. The column ties required for seismic design will be evaluated for the shears induced by the tsunami both in the end section and center section of the column (**Figure D-24**).

Floor 1 – 6

End Section (A)

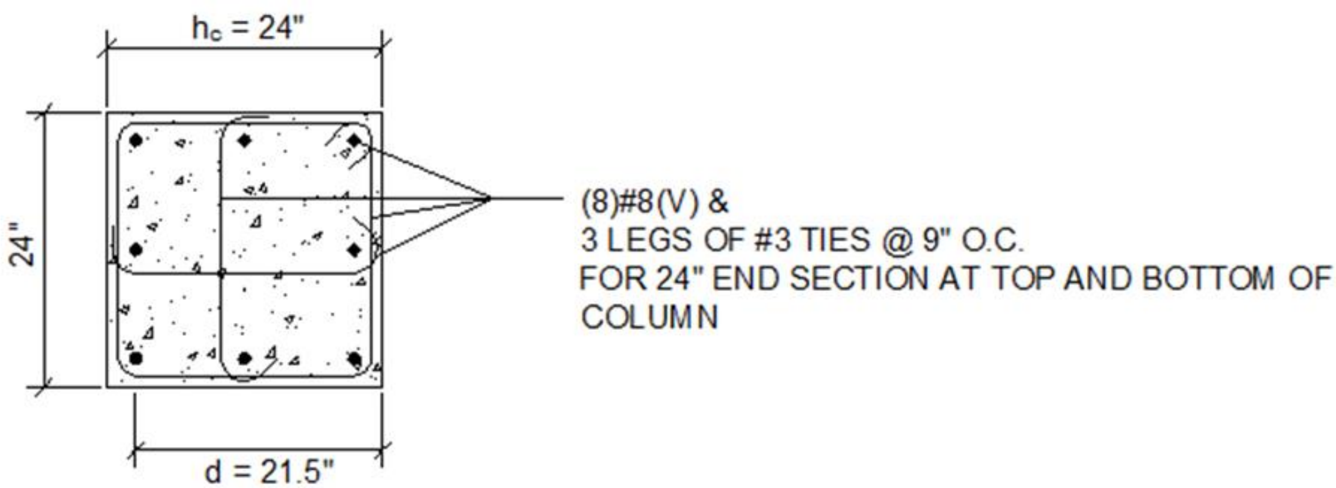


Figure D-22: Exterior column, cross-section at end of column at all floor level based on SDC C design.

Center Section (B)

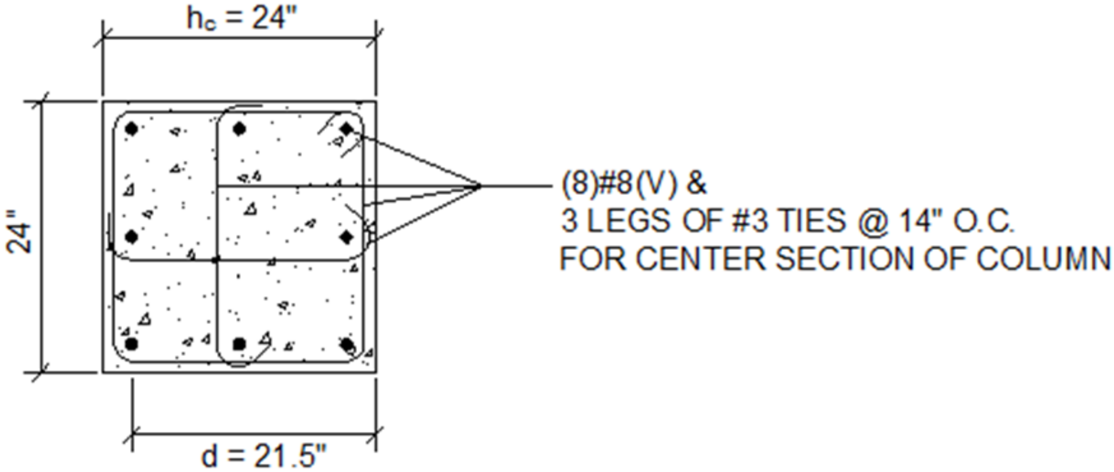


Figure D-23: Exterior column, cross-section at center of column at all floor levels based on SDC C design.

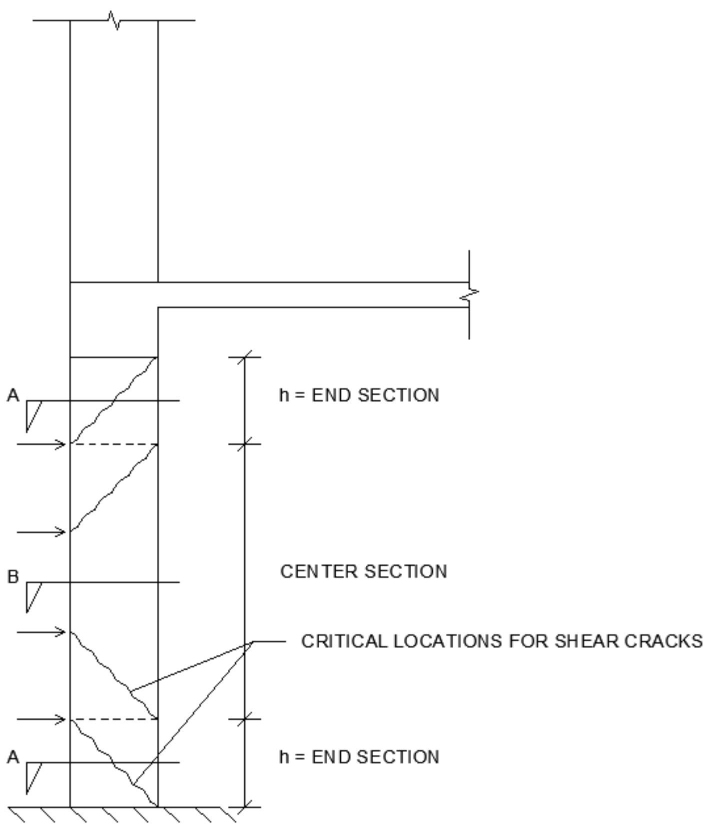


Figure D-24: Typical exterior column elevation showing end and center sections

D.11.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 15 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

P_D = [120(28)(15)(6)+(1.16)(2.5)(150)(28-2.5)+90(28)(5)+2.5^2(150)(74)]/1000 = 395 k

Floor Live load reduction factor = 0.25+15/[4(15)(28)(5)]$^{0.5}$ = 0.414,

therefore, live load at the column base is: P_L = 0.414[65(15)(28)(5)]/1000 = 56.5 k

Roof Live Load reduction factor = R_1R_2= [1.2-(0.001)(15)(28)](1.0) = 0.78,

therefore, column roof live load is: P_{Lr} = 0.78(20)(15)(28)/1000 = 6.55k

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

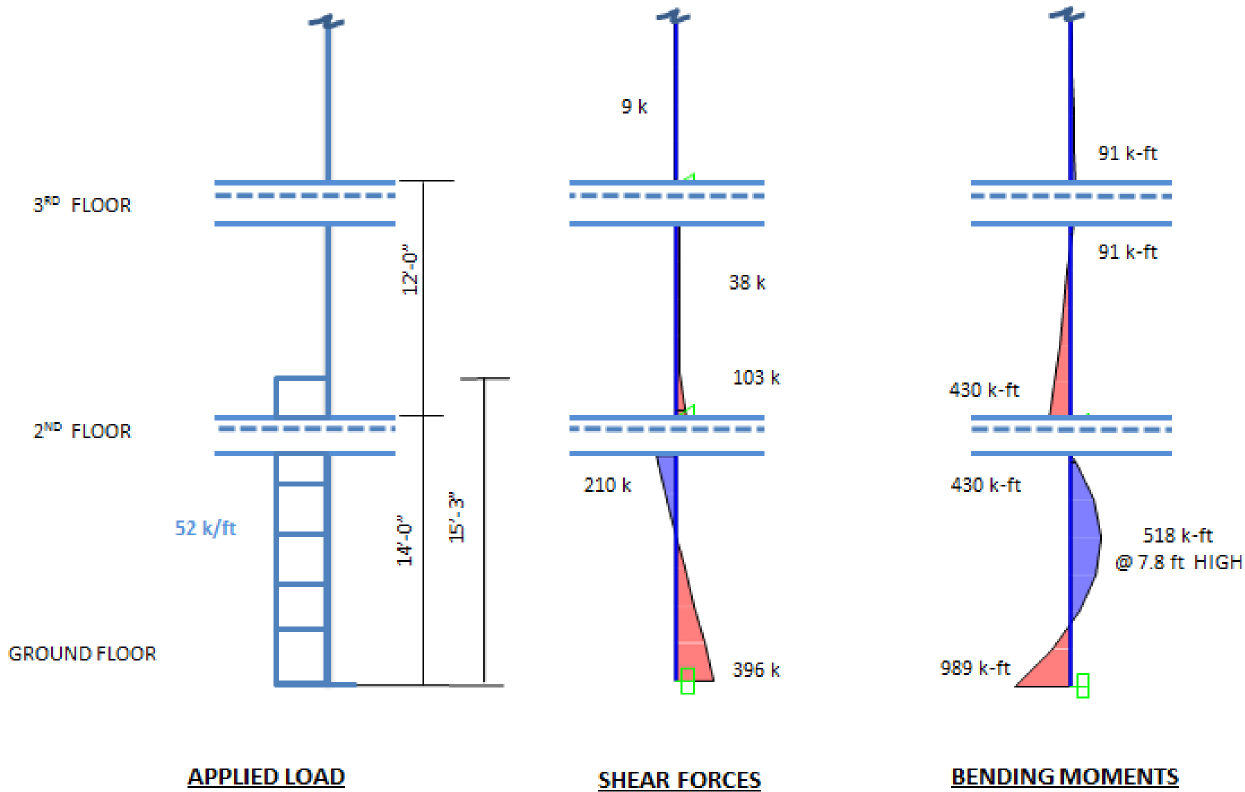


Figure D-25: Hydrodynamic loading on exterior column of the Waikīkī office building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just below the beam at each inundated floor for the maximum shear in the end section of the column. A log strike is also assumed to act just outside the end section (at "d + h_c") and at the mid-height of the clear column height for the maximum shear force and bending moment in the center section, respectively. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure D-26** to **Figure D-27**. The resulting shear force and bending moment diagrams for log impact at a distance "d + h_c" from the end of the column at each floor level are shown in **Figure D-28** to **Figure D-29**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown in **Figure D-30** to **Figure D-31**.

Impact load at d:

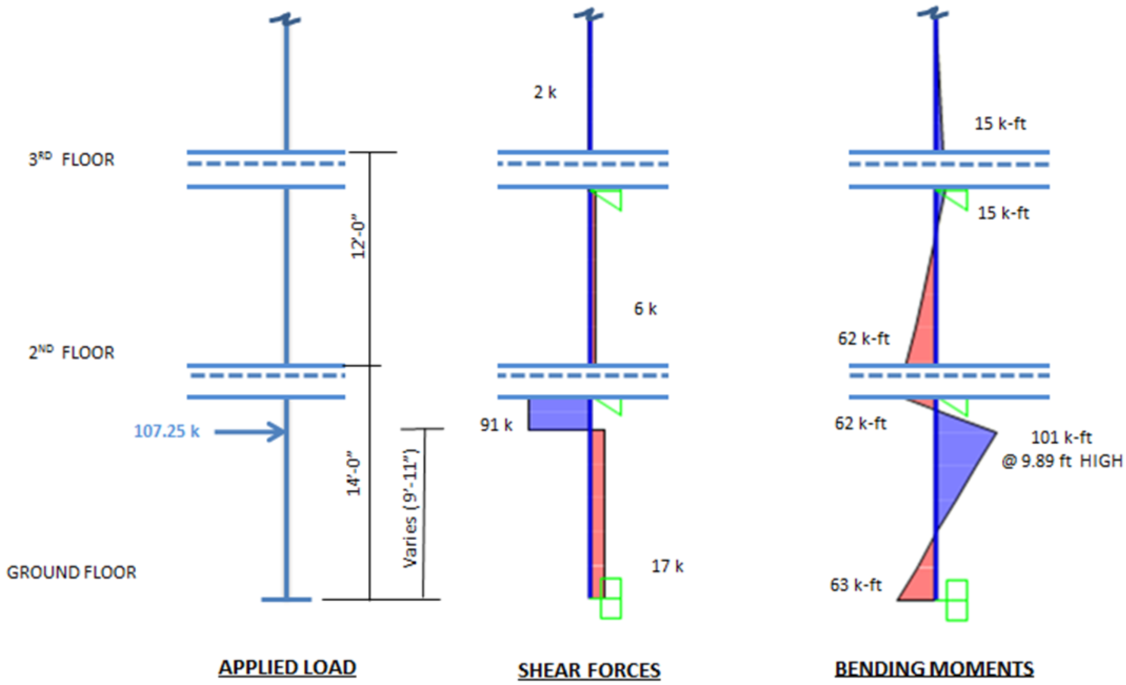


Figure D-26: Impact load applied at "d" away from the end of column on the ground floor

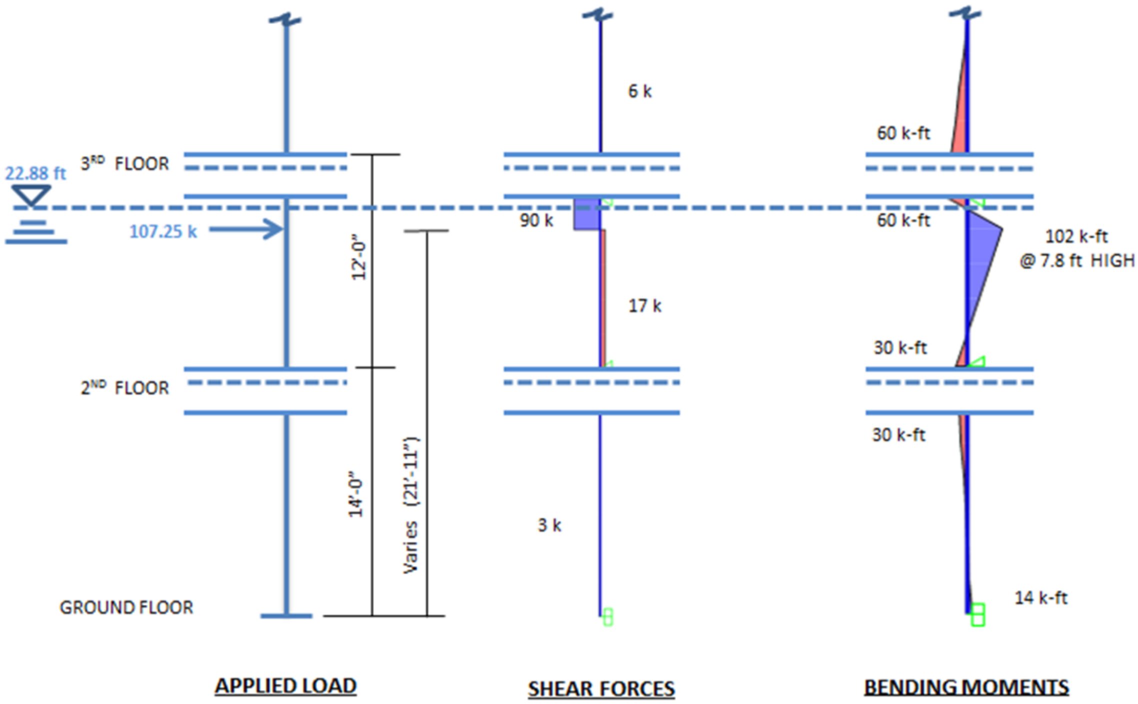


Figure D-27: Impact load applied at "d" away from the end of column on the 2nd floor

Impact load at d + h_c:

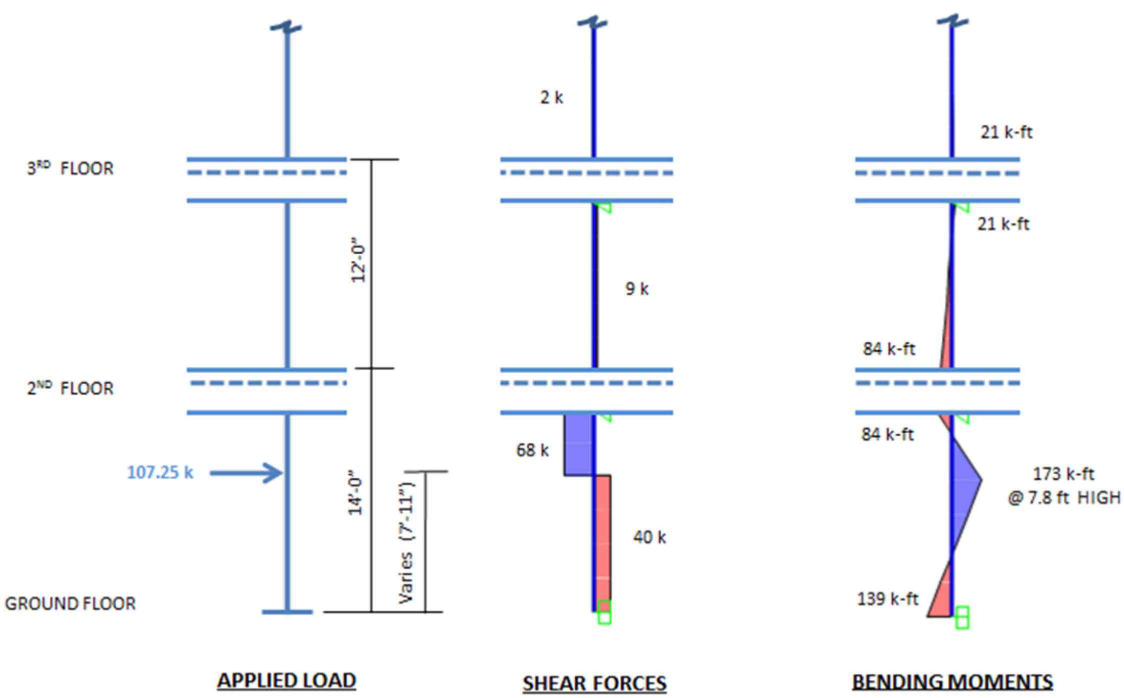


Figure D-28: Impact load applied at "d + h_c" away from the end of column on the ground floor

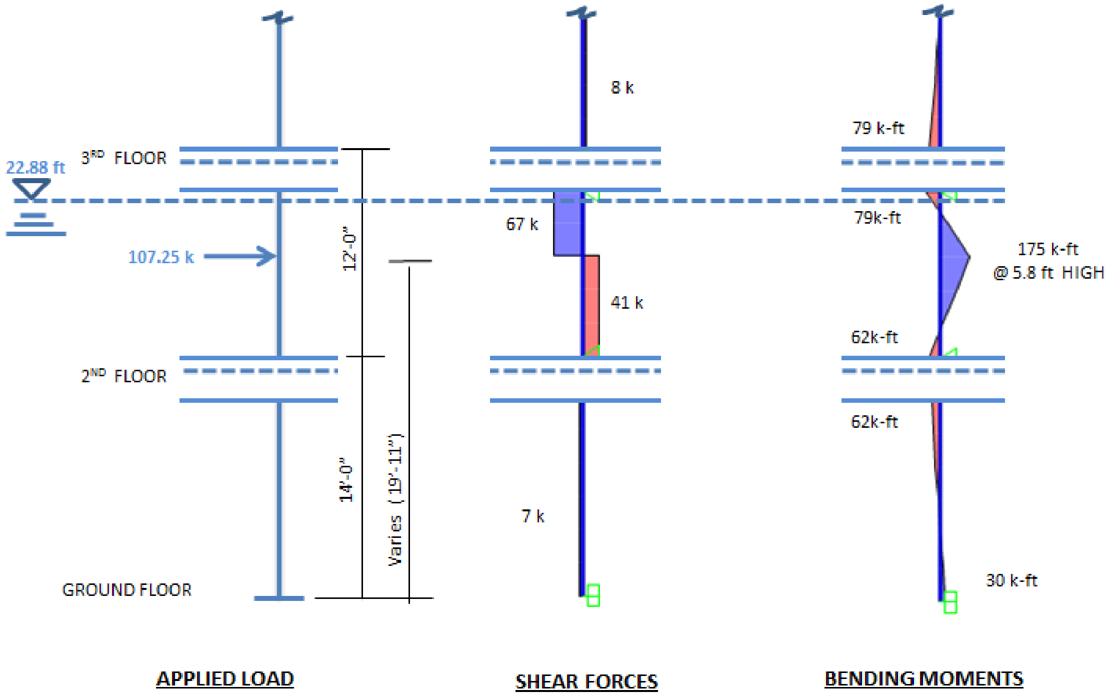


Figure D-29: Impact load applied at "d + h_c" away from the end of column on the 2^{nd} floor

Impact load at mid-height:

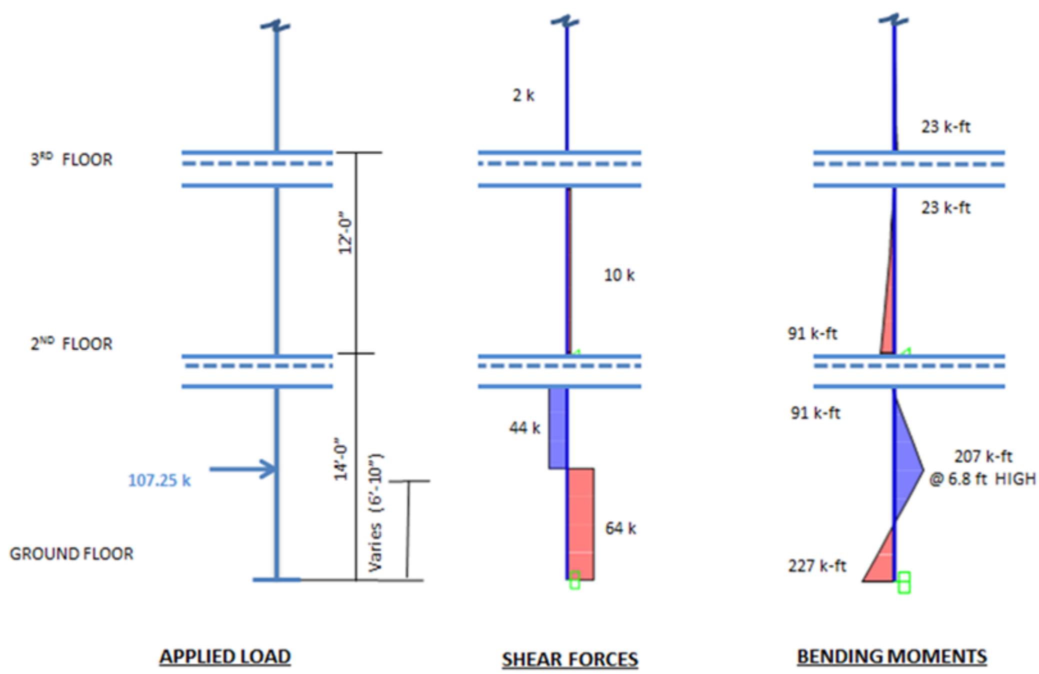


Figure D-30: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

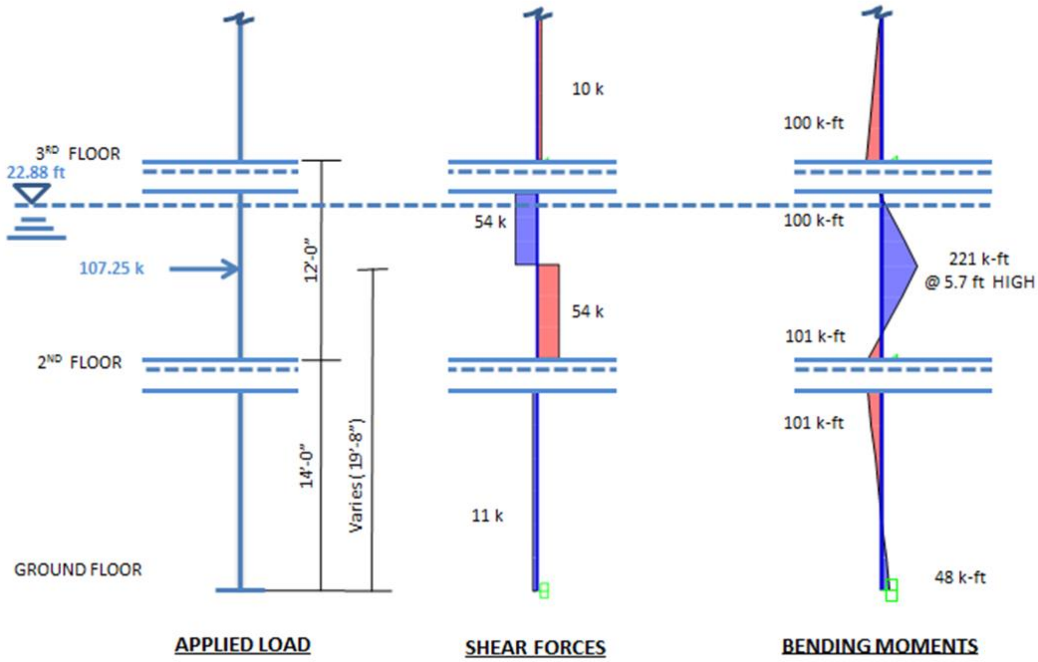


Figure D-31: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

Table D-4 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and log impact (Impact). In addition, because all of the exterior columns are part of the LFRS, Table C-4 also lists the maximum axial load, bending moment and shear forces determined by the ETABS analysis for the modified base shear (Overall) (See Section A.9.3). These "Overall" systemic forces are then combined with the controlling component forces (either "Hydro" or "Impact") to obtain the "Combined" forces. Columns that are part of the transverse MRFs experience larger systemic loads and are therefore considered separately, along with columns having similar loads ("Special").

The original column designs will now be evaluated for these load combinations and modified if necessary.

Table D-4: Results from loading conditions of Waikīkī office building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
989	502.25	303	199	1.2D+Ftsu+0.5L (Hydro)
989	355.5	303	199	0.9D+Ftsu (Hydro)
227	502.25	91	68	1.2D+Ftsu+0.5L (Impact)
227	355.5	91	68	0.9D+Ftsu (Impact)
627	502.25	82	82	1.2D+Ftsu+0.5L (Overall)
627	333.5	82	82	0.9D+Ftsu (Overall)
1504	488.25	385	281	1.2D+Ftsu+0.5L (Combined)
1504	321.5	385	281	0.9D+Ftsu (Combined)
Floor 2				
430	418.54	43	38	1.2D+Ftsu+0.5L (Hydro)
430	296.25	43	38	0.9D+Ftsu (Hydro)
221	418.54	90	67	1.2D+Ftsu+0.5L (Impact)
221	296.25	90	67	0.9D+Ftsu (Impact)
140	418.54	11	11	1.2D+Ftsu+0.5L (Overall)
140	296.25	11	11	0.9D+Ftsu (Overall)
570	418.57	101	78	1.2D+Ftsu+0.5L (Combined)
570	296.25	101	78	0.9D+Ftsu (Combined)
Floor 3				
91	334.83	9	9	1.2D+Ftsu+0.5L (Hydro)
91	237	9	9	0.9D+Ftsu (Hydro)
100	334.83	10	10	1.2D+Ftsu+0.5L (Impact)
100	237	10	10	0.9D+Ftsu (Impact)
Floor 4				
23	251.13	2	2	1.2D+Ftsu+0.5L (Hydro)
23	177.75	2	2	0.9D+Ftsu (Hydro)
25	251.13	3	3	1.2D+Ftsu+0.5L (Impact)
25	177.75	3	3	0.9D+Ftsu (Impact)
Floor 5				
6	167.42	1	1	1.2D+Ftsu+0.5L (Hydro)
6	118.5	1	1	0.9D+Ftsu (Hydro)
5	167.42	1	1	1.2D+Ftsu+0.5L (Impact)
5	118.5	1	1	0.9D+Ftsu (Impact)
Floor 6				
1	83.71	0	0	1.2D+Ftsu+0.5L (Hydro)
1	59.25	0	0	0.9D+Ftsu (Hydro)
2	83.71	0	0	1.2D+Ftsu+0.5L (Impact)
2	59.25	0	0	0.9D+Ftsu (Impact)

D.11.1.2 Existing Exterior Column Design for Combined Gravity and Tsunami Loads

The column chosen at Grid Intersection A-3 from **Figure D-15** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure D-32 to **Figure D-34** shows the interaction diagram for the typical exterior column including the tsunami load combinations.

The blue solid line (Original Column Design Strength) represents the design strength for the original columns. The green dashed line (New Column Design Strength) represents the design strength needed if one were to take into account only the hydrodynamic and impact loads shown in **Figure D-25** to **Figure D-31**. The dotted red line (New Overall Column Design Strength) represents the design strength needed for taking into account only the overall building forces for each column shown in **Figure D-20** to **Figure D-21.** The orange dot-dashed line (New Combined Column Design Strength) represents the design strength needed for the overall loading combined with the hydrodynamic and impact loads per column. This series of plots is shown in alternating figures from **Figure D-32** to **Figure D-36** for all affected floor levels**.** Alternating **Figure D-33** to **Figure D-35** show the interaction diagrams for the combined forces with the controlling load combination for each column.

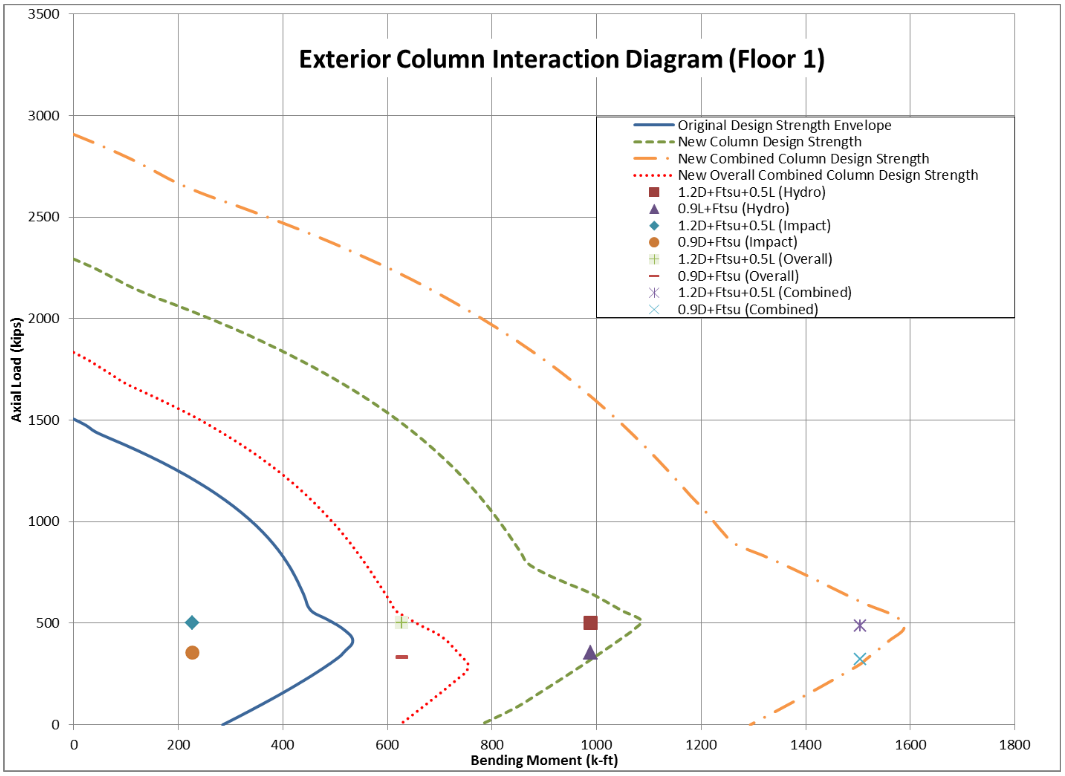


Figure D-32: Sequence of interaction diagrams for typical ground floor exterior column showing various tsunami load combinations

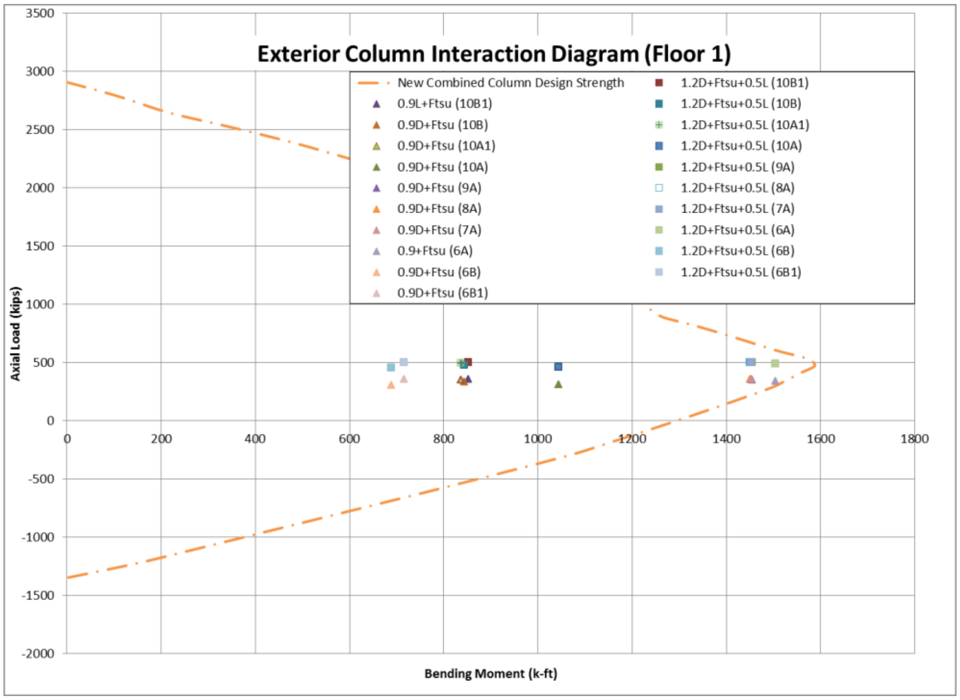


Figure D-33: Interaction diagrams for typical and special ground floor exterior column showing all combined tsunami load combinations

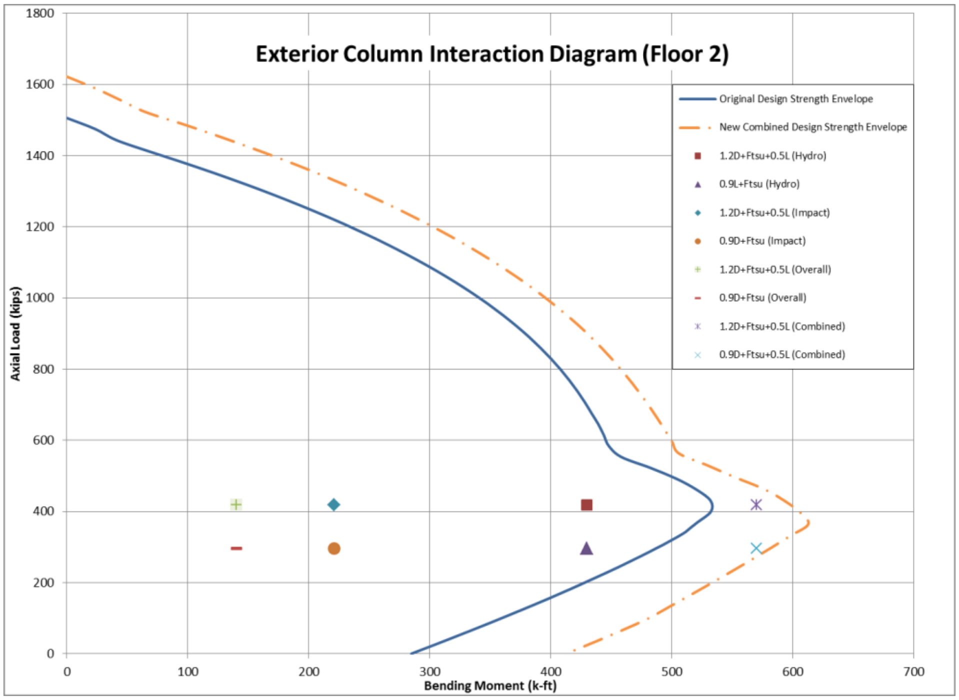


Figure D-34: Sequence of interaction diagrams for typical 2nd floor exterior column showing tsunami load combinations

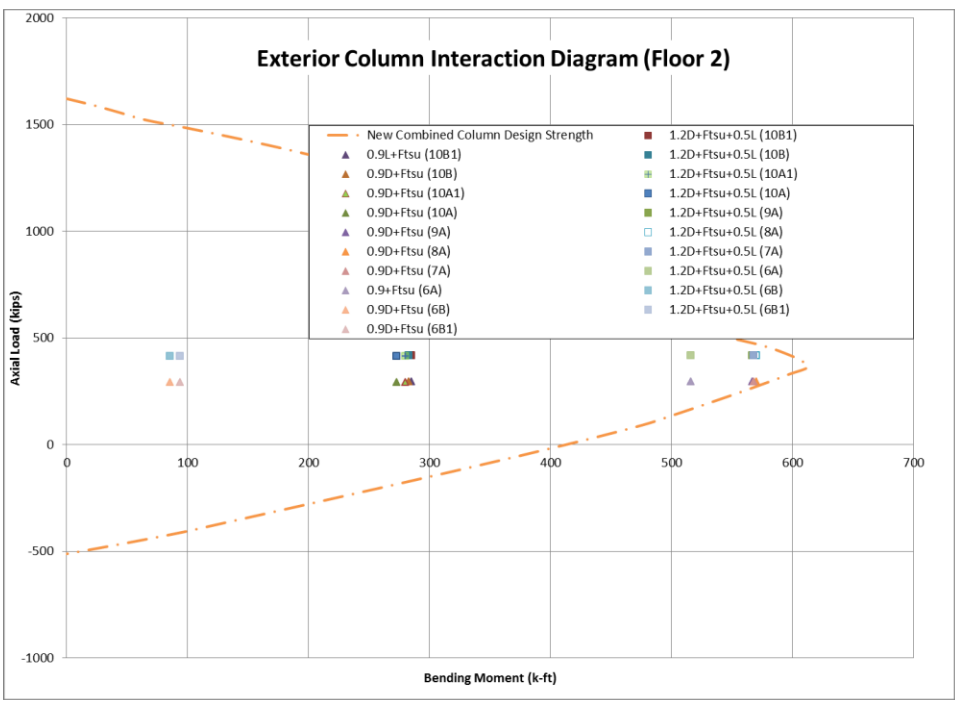


Figure D-35: Interaction diagrams for typical and special 2nd floor exterior column showing all combined tsunami load combinations

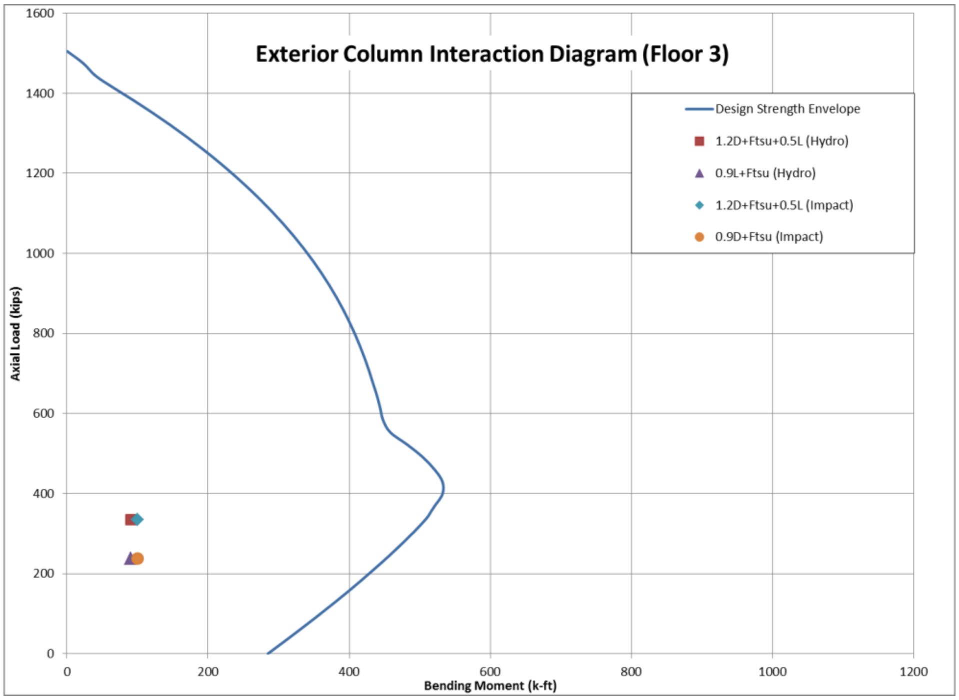


Figure D-36: Sequence of interaction diagrams for typical 3rd floor exterior column showing tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

D.11.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure D-32** to **Figure D-34** the original exterior columns are adequate for log impact load, but the columns at the ground floor must be strengthened to resist bending due to the hydrodynamic and overall system loads. Revised column designs shown in **Figure D-37** to **Figure D-38** were developed to satisfy the combined hydrodynamic and overall loads. The interaction diagram for this new column is shown in **Figure D-32**. The ties in these columns are designed in Section A.11.1.4 for the applied tsunami shear forces.

Floor 1

End Section (A)

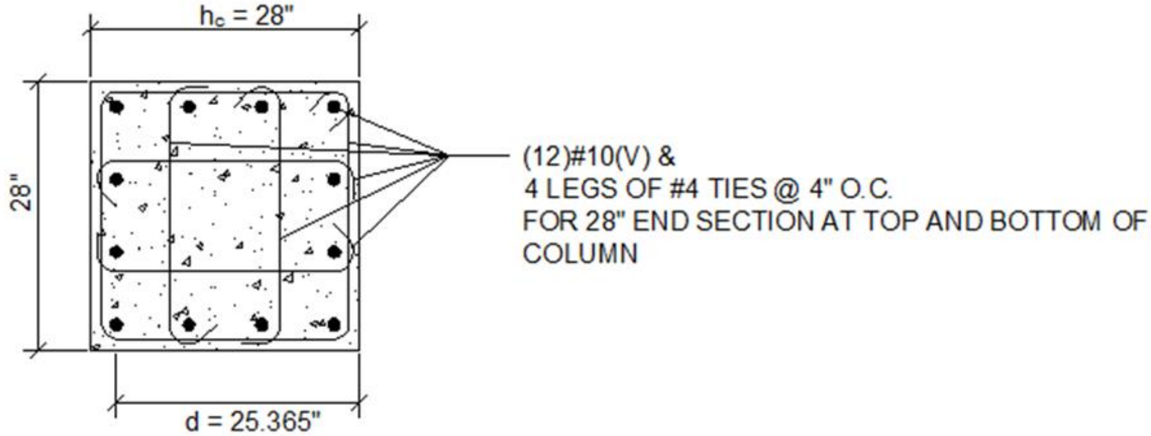


Figure D-37: Exterior column, cross-section at end section of column at ground floor level based on tsunami design requirements.

Center Section (B)

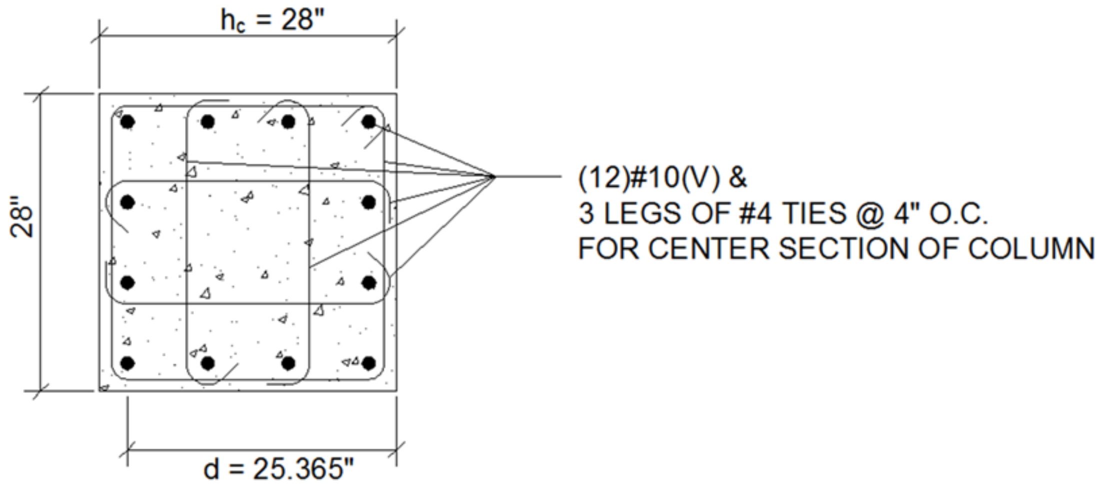


Figure D-38: Exterior column, cross-section at center section of column at ground floor level based on tsunami design requirements.

D.11.1.4 Exterior Column Shear Design

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 321.5 kips.

The shear capacities of the 30”x30” columns with 4 leg #5 Stirrups at 4” o.c. in the end section and 4 leg #4 Stirrups at 4” o.c. in the center section are given by

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{321{,}500}{2{,}000\times30\times30}\right)30\times27.295/1{,}000$ = 122 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.31)\times60{,}000\times27.295}{4\times1{,}000}$ = 508 kips

$V_S = \frac{A_v f_y d}{s}$ = 508 kips $\ngtr 8\sqrt{f_C'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}000}\times30\times27.295 = 414$ kips $\therefore$ use 414 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times27.295}{4\times1{,}000}$ = 328 kips

$V_S = \frac{A_v f_y d}{s}$ = 328 kips $\ngtr 8\sqrt{f_C'}\,\mathrm{b\,d} = 8\times\sqrt{4{,}000}\times30\times27.295 = 414$ kips $\therefore$ use 328 kips

Therefore in the end sections, $\text{Ø}V_n$ = 0.75 (122 + 414) = 402 k

and in the center sections, $\text{Ø}V_n$ = 0.75 (122 + 328) = 337 k

At d: V_u = 385 k < ϕV_n = 402 k, therefore the column is adequate for shear at the edge.

At $d + h_c$: V_u = 281 k < ϕV_n = 337 k, therefore the column is adequate for shear at the center.

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b,** P_u = 296.25 k:

The shear capacities of the 24”x24” column with 4 leg #3 stirrups at 9” o.c. in the end sections and 3 leg #3 stirrups at 14” o.c. in the center section are given by:

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{296{,}250}{2{,}000\times24\times24}\right)24\times21.5/1{,}000$ = 82 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times21.5}{9\times1{,}000}$ = 63 kips

$$V_S = \frac{A_v f_y d}{s} = 63 \text{ kips} \ngtr 8\sqrt{f_C{}'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 24 \times 21.5 = 261 \text{ kips} \therefore \text{use 63 kips}$$

and in the center sections, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times21.5}{14\times1{,}000} = 30$ kips

$$V_S = \frac{A_v f_y d}{s} = 30 \text{ kips} \ngtr 8\sqrt{f_C{}'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 24 \times 21.5 = 261 \text{ kips} \therefore \text{use 30 kips}$$

therefore (End; 3 leg #4 Stirrups @ 4" o.c.) $ØV_n$ = 0.75 (82 + 63) = 109 k

therefore (Center; 3 leg #3 Stirrups @ 6" o.c.) $ØV_n$ = 0.75 (82 + 30) = 63 k

At d: V_u = 101 k < ϕV_n = 109 k, therefore the column is adequate for shear at the edge.

At $d + h_c$: V_u = 78 k < ϕV_n = 84 k, therefore the column is adequate for shear at the center.

By inspection the remaining columns are adequate to resist the tsunami shear force.

D.11.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure D-15**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category C. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 24 in square column cross section shown in **Figure D-39** and **Figure D-40** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force for the end section of the column occurs at a distance "*d*" from the ends of the column, where *d* = 24 - 1.5 - 0.5 - 0.5 = 21.5 in. The critical shear force for the center section of the column occurs at "$d + h_c$" from the end of the column, where $d + h_c$ = 21.5 + 24 = 45.5 in.

Floor 1 – 6

End Section (A)

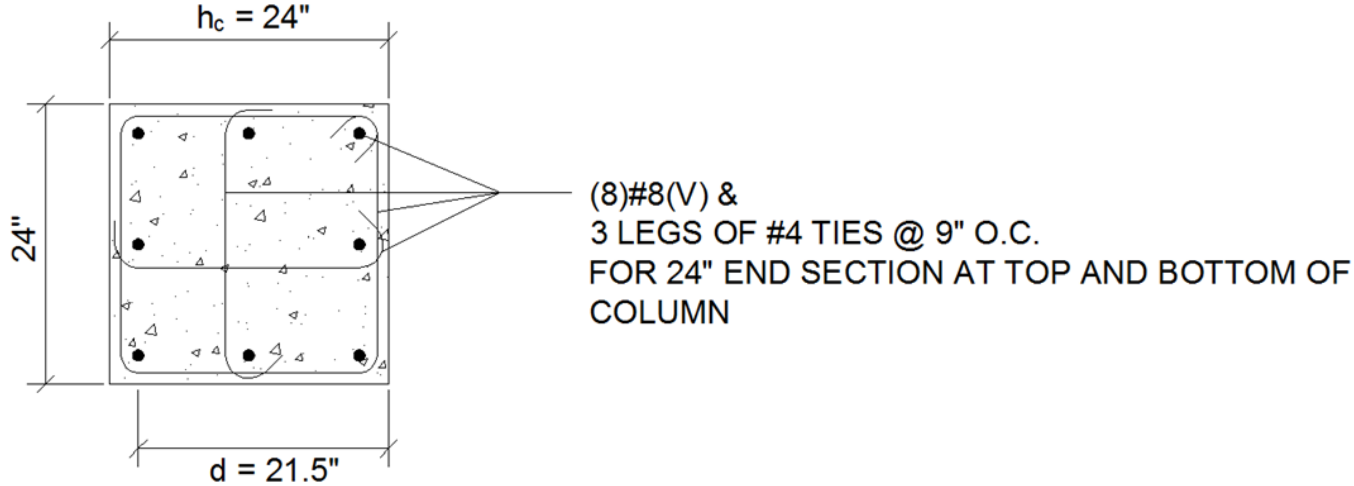


Figure D-39: Interior column, end section cross-section for column at all floor levels based on SDC D design.

Center Section (B)

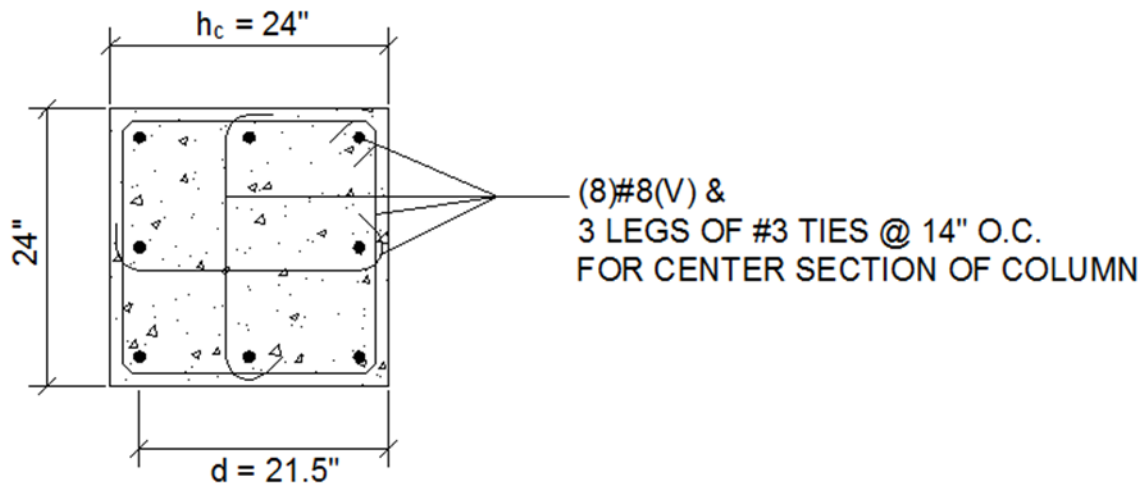


Figure D-40: Interior column, center section cross-section for column at all floor levels based on SDC D design.

D.11.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 29 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$$P_D = [120(28)(29)(6) + 2^2(150)(74)]/1000 = 629\ k.$$

Floor Live load reduction factor = $0.25+15/[4(29)(28)(5)]^{0.5} = 0.367$*, therefore using 0.4 gives:*

$$P_L = 0.4[95(5)+65(24)](28)(5)]/1000 = 114\ k.$$

Roof Live Load reduction factor = $R_1R_2 = 0.6(1.0) = 0.6$ *for* $A_t > 600$ *sf, therefore the roof live load is:*

$P_{Lr} = 0.6(20)(28)(29) = 9.7\ k$

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

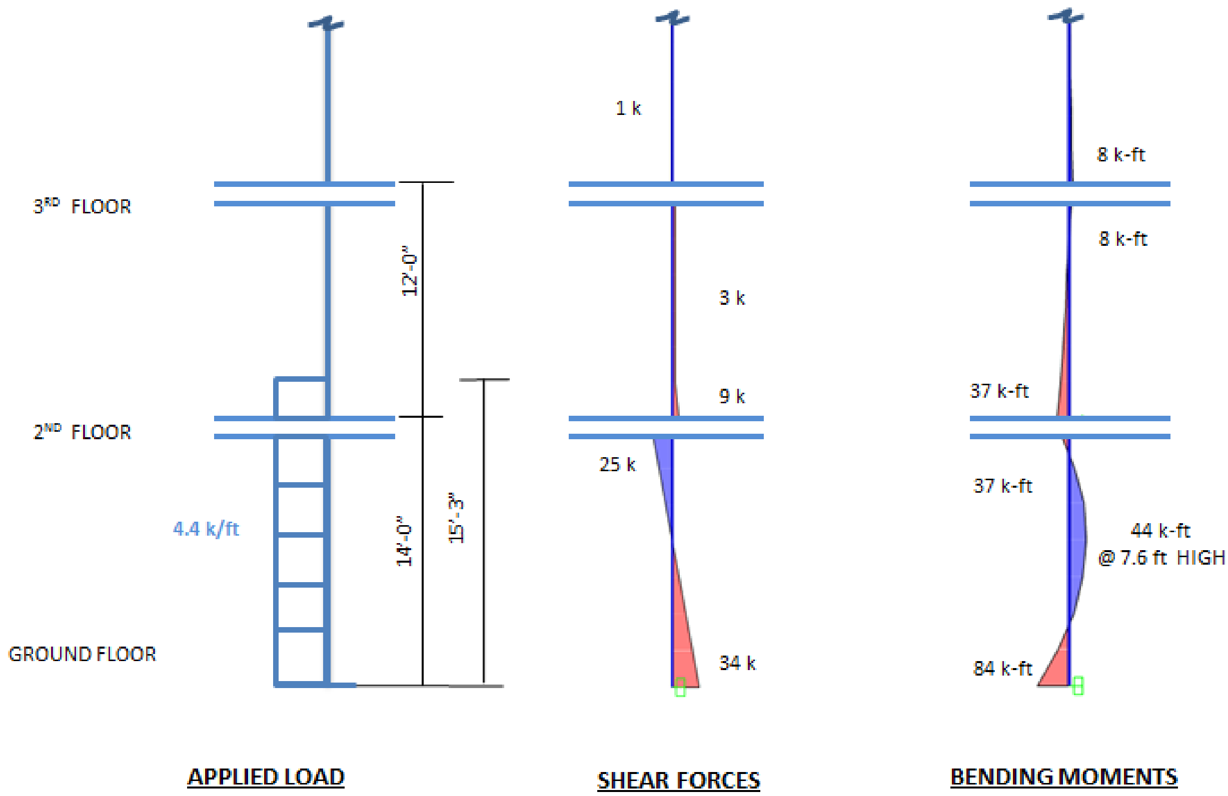


Figure D-41: Hydrodynamic loading on interior column of Hilo office building due to Load Case 2

Table D-5 summarizes the maximum axial load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table D-5: Results from loading conditions of Waikīkī office building interior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
84	811.8	26	17	1.2D+Ftsu+0.5L (Hydro)
84	566.1	26	17	0.9D+Ftsu (Hydro)
Floor 2				
37	676.5	4	3	1.2D+Ftsu+0.5L (Hydro)
37	471.75	4	3	0.9D+Ftsu (Hydro)
Floor 3				
8	541.2	1	1	1.2D+Ftsu+0.5L (Hydro)
8	377.4	1	1	0.9D+Ftsu (Hydro)
Floor 4				
2	405.9	0	0	1.2D+Ftsu+0.5L (Hydro)
2	283.05	0	0	0.9D+Ftsu (Hydro)
Floor 5				
0	270.6	0	0	1.2D+Ftsu+0.5L (Hydro)
0	188.7	0	0	0.9D+Ftsu (Hydro)
Floor 6				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.35	0	0	0.9D+Ftsu (Hydro)

D.11.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure D-15** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure D-42 shows the interaction diagram for a typical interior column with the tsunami load combinations.

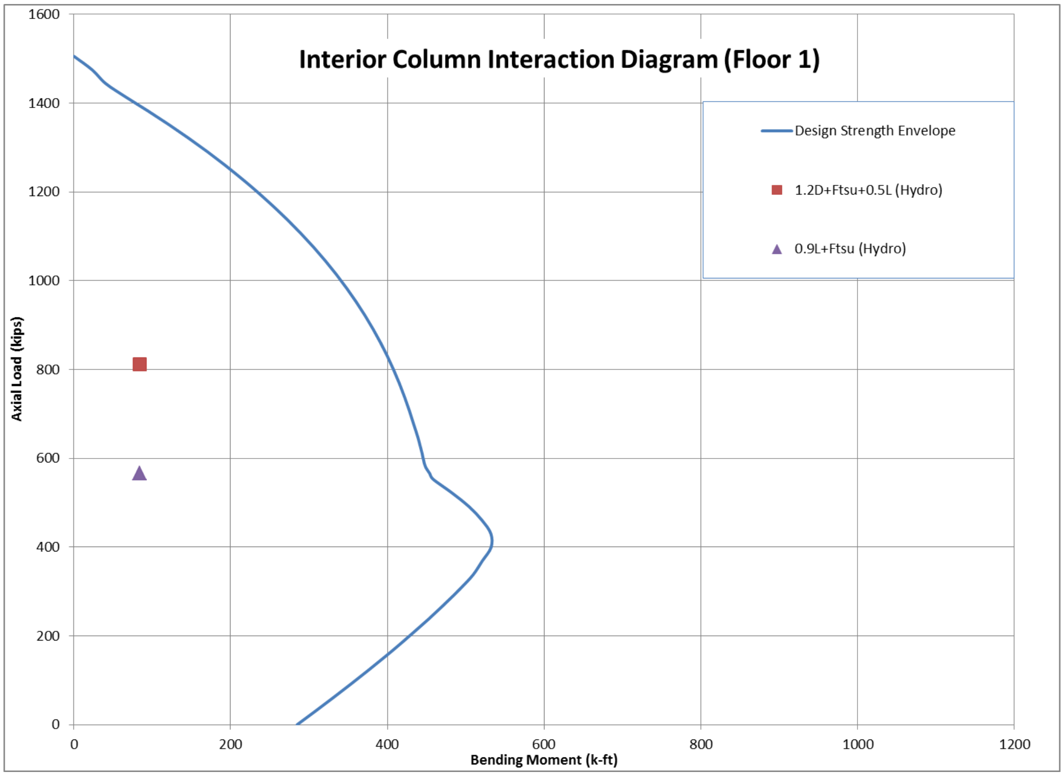


Figure D-42: Interaction diagram for typical ground floor office interior column showing tsunami load combinations

The existing interior column is therefore adequate at the first floor level, and by inspection the remaining columns are also adequate to resist the tsunami bending moments.

D.11.2.3 Interior Column Shear Design

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of (1.2D + F_{TSU} + 0.5L) per **Eqn. 6.8.3.3.-1a,** P_u = 812 kips.

The shear capacities of the existing 24”x24” column with 3 leg #4 Stirrups @ 9” o.c. in the end sections and 3 leg #3 Stirrups @ 14” o.c. in the center section are given by:

$\emptyset V_n = \emptyset(V_C + V_S)$

where $V_C = 2\sqrt{f_C{}'}\left(1+\frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1+\frac{811{,}800}{2{,}000\times24\times24}\right)24\times21.5/1{,}000$ = 111 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times21.5}{9\times1{,}000}$ = 30 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times21.5}{14\times1{,}000}$ = 86 kips

Therefore in the end sections, $\emptyset V_n$ = 0.75 (111 + 86) = 148 k

and in the center sections, $\emptyset V_n$ = 0.75 (111 + 30) = 106 k

At d: V_u = 43 k < ϕV_n = 148 k, therefore the column is adequate for shear at the edge.

At d + h_c: V_u = 26 k < ϕV_n = 106 k, therefore the column is adequate for shear at the center.

By inspection the remaining columns are adequate to resist the tsunami shear force.

D.12 Tsunami Design for Residential Building

D.12.1 Simplified Equivalent Uniform Lateral Static Force – Section 6.10.1 (Optional)

In lieu of performing detailed hydrostatic and hydrodynamic analysis, ***Eqn. 6.10.1-1*** *provides a simplified but conservative estimate of the maximum lateral load on the building.*

$$f_{uw} = 2.5 I_{tsu} \gamma_s h_{max}^2 = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 22.88^2 = 92.15\ kip/ft$$

Assuming $C_{cx} = 0.7$ *controls per* ***Section 6.8.7***

Therefore $\quad F = 0.7 \times 254 \times 92.15 = 16{,}384\ kips$

This lateral load can be compared with 0.75 $\Omega_o E_h$ *= 0.75 x 2.5 x 1,565 = 2,934 kips < 16,384 kips. Therefore the LFRS is not adequate to satisfy this requirement and the detailed analysis for LC2 and LC3 shown below is recommended. The components can also be designed on the basis of this conservative uniform distributed force with the appropriate width b dimensions (but that is not illustrated here).*

D.12.2 Overall Building Forces

Section 6.8.3.1 defines the following three Load Cases, which must be considered in the design.

D.12.2.1 Load Case 1: Maximum buoyancy and associated hydrodynamic drag

The exterior inundation depth need not exceed the lesser of

h_{ext} $\quad \leq h_{max}$ = 22.88 ft

$\leq$ 10 ft

$\leq$ top of first story windows = 8 ft. CONTROLS

Because the ground floor consists of a slab-on-grade that is isolated from the building columns, any uplift pressures developed below the slab will cause localized slab failure but will not result in buoyancy of the building. Therefore overall buoyancy is not a consideration.

Load Case 1 also requires application of the associated hydrodynamic drag on the entire building. However this will not control since buoyancy need not be considered.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2} \rho_s I_{tsu} C_d C_{cx} B (hu^2)$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft

I_{tsu} = 1.0 (**Table 6.8-1** – TRC II)

C_d = 1.4575 (**Table 6.10-1** based on B/h_{sx} = 254/8 = 31.8)

$C_{cx} = 1.0$ since the exterior walls are assumed to be intact for Load Case 1

B = 254’ overall width of building

h = 8’

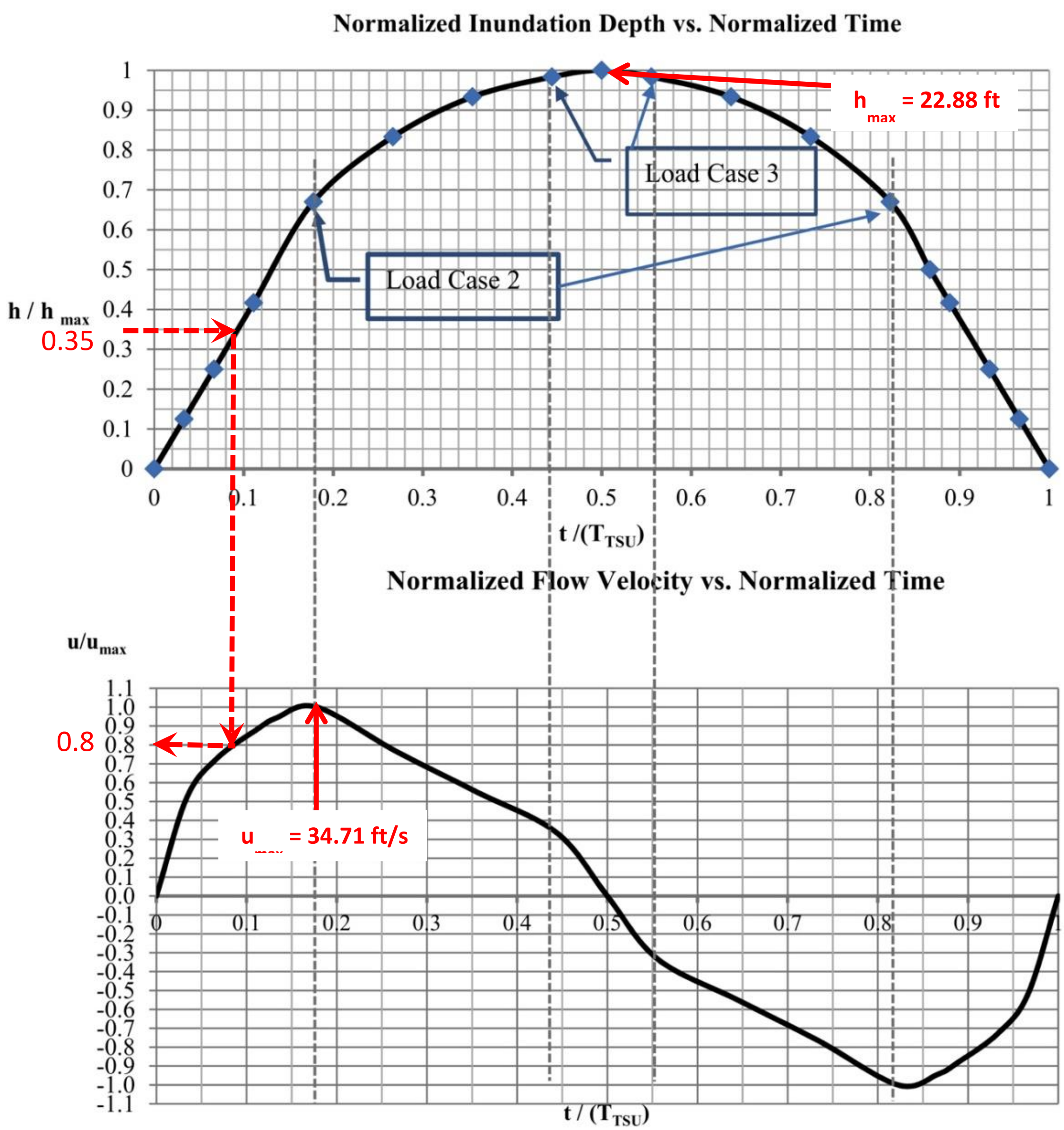


Figure D-43: Determining “u” for LC1 with ASCE 7-16 Figure 6.8-1

Figure 6.8-1 is used to determine the flow velocity corresponding to an inundation depth of 8 ft. For h = 8', h/h_{max} = 8/22.88 = 0.35. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.09. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.8. Therefore the flow velocity is u = 0.8 x 34.71 = 27.77 fps.

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4575 \times 1.0 \times 254(8 \times 27.77^2)/1000 = 2{,}512\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8 feet above grade. The lateral force resisting system for the structure would be evaluated for this load. The non-structural exterior wall should not be designed for this load since it is preferred that non-structural walls fail to relieve lateral load on the structural frame. Note that a portion of this load will go to the ground floor slab, which reduces the load that has to be resisted by the lateral force resisting system. The entire lateral load must be resisted by the deep foundations assuming maximum scour has already occurred.

D.12.2.2 Load Case 2: Maximum Flow Velocity

According to **Figure 6.8-1**, LC2 occurs when the inundation depth is 2/3h_{max} = 2/3 x 22.88 = 15.25 ft.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.306 (**Table 6.10-1** based on B/h_{sx} = 254/15.25 = 16.65)

Since the inundation depth of 15.25 feet exceeds the bottom of the second floor slab (12' – 8"/12) = 11.33', the inundated area of the beams must be included in the closure coefficient, which is determined as follows:

h_{sx} = 15.25 ft.

A_{col} = 32 x 1.67' x (15.25'-0.67') = 779 ft^2

A_{wall} = 2 x 28' x (15.25'-0.67') + 2 x 10' x (15.25'-0.67') = 1109 ft^2

A_{Beam} = A_{Slab} = 1 x 254' x 0.67' = 169 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5\times A_{beam}}{Bh_{sx}} = \frac{\sum((779+1109)+1.5\times 169}{254\times 15.25} = 0.552 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

$u = u_{max}$ = 34.71 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.306 \times 0.7 \times 254(15.25 \times 34.71^2)/1000 = 4{,}697\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal and inland elevations of the building over a height of 15.25 feet above grade. The lateral force resisting system for the structure must be evaluated for this load. During drawdown the same pressure needs to be applied to the inland elevation and the lateral force resisting system evaluated for this load.

D.12.2.3 Load Case 3: Maximum Inundation Depth

According to **Figure 6.8-1**, LC3 occurs when the inundation depth is h_{max} = 22.88 ft. and the flow velocity is 1/3u_{max} = 1/3 x 34.72 = 11.57 fps.

Section 6.10.2, Eqn. 6.10.2-1 gives $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$

Where all parameters are the same as for LC1 except:

C_d = 1.3 (**Table 6.10-1** based on B/h_{sx} = 254/22.88 = 11.10)

Since the inundation depth of 22.88 ft exceeds the third floor slab elevation of 21 ft, the closure coefficient is given by:

h_{sx} = 22.88 ft.

A_{col} = 32 x 1.67' x (22.88'-0.67'-0.67') = 1150 ft^2

A_{wall} = 2 x 28' x (22.88'-0.67'-0.67') + 2 x 10' x (22.88'-0.67'-0.67') = 1638 ft^2

A_{Beam} = A_{Slab} = 2 x 254' x 0.67' = 339 ft^2

$$C_{cx} = \frac{\sum(A_{col}+A_{wall})+1.5A_{beam}}{Bh_{sx}} = \frac{\sum(1150+1638)+1.5\times 339}{254\prime \times 22.88\prime} = 0.567 < 0.7$$

Therefore $C_{cx} = 0.7$ controls per **Section 6.8.7**

u = 11.57 fps

So $F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.25 \times 0.7 \times 254(22.88 \times 11.57^2)/1000 = 749\ kips$

This load would be applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 22.88 feet above grade. Although LC3 does not control design of the lateral force resisting system, the intent of LC3 is to ensure evaluation of components up to the maximum inundation depth.

D.12.2.4 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category C, **Section 6.8.3.4** permits the use of 0.75$\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear. From the seismic design of this structure, E_h = 1,565 kips. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 2.5 \times 1{,}565 = 2{,}934\ kips$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 4,697 kips applied over a height of 15.25 ft. A portion of this load will be resisted by the grade beam/foundation system, reducing the overall load by 1,848 kips. (**Figure D-44**)

$$0.75\Omega_o E_h = 2{,}934\ kips > 2{,}849\ kips$$

Therefor the lateral force resisting system has ample capacity to resist the overall tsunami loads.

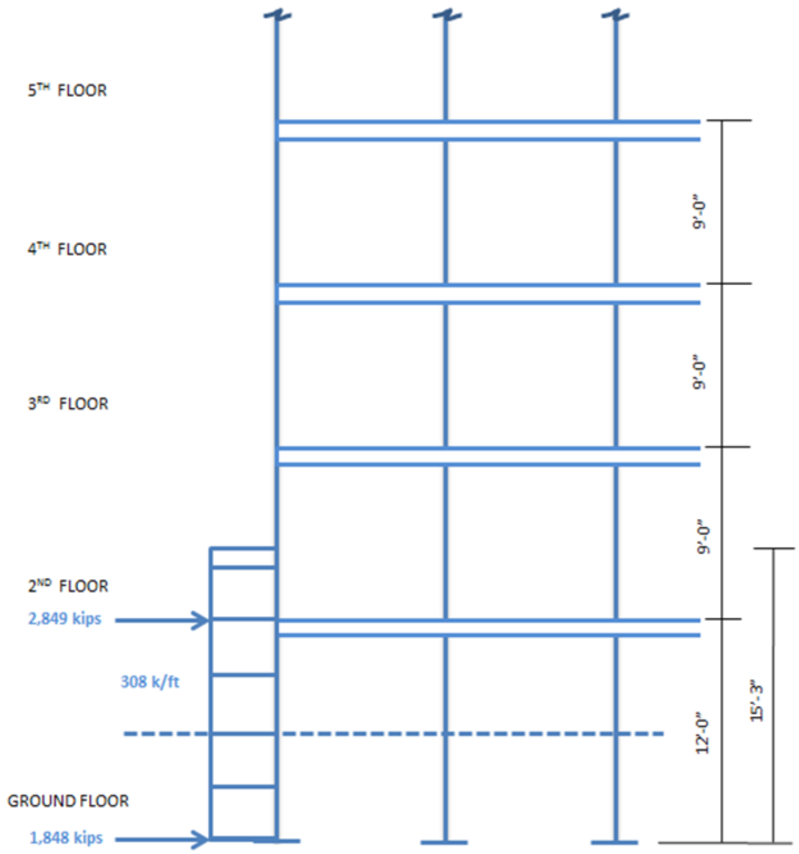


Figure D-44: LC2 Tsunami loads on overall Waikīkī Residential building

D.13 Component Design

D.13.1 Drag Force on Components - Section 6.10.2.2

D.13.1.1 Exterior Columns

Exterior columns are assumed to have accumulated debris resulting in an increased tributary width for hydrodynamic load. **Section 6.10.2.2** requires that C_d = 2.0 and the width dimension, b, be taken as the tributary width multiplied by the closure ratio value, C_{cx}, given in **Section 6.8.7**. Therefore b = 0.70x28' = 19.6 ft.

The controlling load case will be LC2, when the inundation depth is h_e = 15.25 ft and u_{max} = 34.71 fps.

The hydrodynamic drag is computed using **Eqn 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(15.25 \times 34.71^2)/1000 = 793\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 793/15.25 = 51.96 kips/ft over the lower 15.25 feet of the column. The column must be designed for this load combined with gravity loads per **Section 6.8.3.3**.

D.13.1.2 Interior Columns

Interior columns are 20" (1.67 ft) square R.C. columns. The controlling load case will be LC2, when the inundation depth is is h_e = 15.25 ft and u_{max} = 34.71 fps.

The hydrodynamic drag is computed using **Eqn. 6.10.2-3** as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$$

Where C_d = 2.0 for square columns (**Table 6.10-2**) and b = 1.67 ft since no debris accumulation is considered for interior column.

Therefore $F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 1.67(15.25 \times 34.71^2)/1000 = 67.4\ kips$

This load is applied to the column as an equivalent uniformly distributed lateral load of 67.4/15.25 = 4.42 kips/ft over the lower 15.25 feet of the column. This load must be combined with gravity loads per **Section 6.8.3.3** and the column capacity verified.

D.13.2 Tsunami Loads on Structural Walls, F_w – Section 6.10.2.3

Since tsunami bores are anticipated at this location, the lateral load on the structural walls is given by **Eqn. 6.10-5a** or **Eqn. 6.10-5b**, depending on the flow depth relative to the wall width:

Eqn. 6.10-5a $F_W = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2)$

Eqn. 6.10-5b $F_W = \frac{3}{4}\rho_s I_{tsu} C_d b(h_e u^2)$ when $\frac{b_w}{h_e} \geq 3$

Where C_d = 2.0 for a wall per **Table 6.10-2**, and

Elevator Walls:

b = 28' for the elevator walls

Elevator $\frac{b_w}{h_e} = \frac{28'}{15.25'} = 1.84 \ngeq 3 \therefore \text{Eqn. 6.10-5a}$

The controlling load case will be LC2, where h_e = 15.25 ft and u = 34.71 fps.

Therefore, for the 28' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 28(15.25 \times 34.71)/1000 = 1{,}132\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 1,132/(28x15.25) = 2,651 psf over the lower 15.25 ft of the walls. This will not govern when compared with the pressure from hydrodynamic drag in Eqn. 6.10-5b.

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3} \rightarrow h_e \geq \frac{b_w}{3'} = \frac{28'}{3'}$ = 9.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 9.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure D-45**. The resulting velocity is h/h_{max} = 9.33'/22.88' = 0.408. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at $t/(T_{TSU})$ = 0.11. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.84. Therefore the flow velocity is u = 0.84 x 34.71 = 29.16 fps. The bore loading is computed for h_e = 9.33 ft and u = 29.16 fps.

Therefore for the 28' wide elevator wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 28(9.33 \times 29.16^2)/1000 = 733\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 733/(28x9.33) = 2,806 psf over the lower 9.33 ft of the walls. ← **(CONTROLS)**

Stairwell Walls:

b = 10' for the elevator walls

Stairwell $\frac{b_w}{h_e} = \frac{10'}{15.25'} = 0.656 \ngeq 3 \therefore \text{Eqn. 6.10-5a}$

The controlling load case will be LC2, where h_e = 15.25 ft and u = 34.71 fps.

Therefore, for the 10' wide elevator wall, $F_d = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 10(15.25 \times 34.71^2)/1000 = 404\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 404/(10x15.25) = 2,649 psf over the lower 15.25 ft of the walls. This will govern when compared with the pressure from hydrodynamic drag in Eqn. 6.10-5b.

It is possible that the inundation occurs as a series of bores each with height less than h_{max}. In this case, a critical bore height would be $\frac{h_e}{b_w} \geq \frac{1}{3} \rightarrow h_e \geq \frac{b_w}{3'} = \frac{10'}{3'}$ = 3.33', since this would require consideration of **Eqn. 6.10-5b.** For a flow depth of 3.33 ft, the flow velocity can be obtained from ASCE 7-16 **Figure 6.8-1**, as shown in **Figure D-45**. The resulting velocity is h/h_{max} = 3.33'/22.88' = 0.146. Identifying this point on the inflow side of **Figure 6.8-1(a)** indicates that this inundation depth occurs at

$t/(T_{TSU})$ = 0.03. At the same time in **Figure 6.8-1(b)** the flow velocity ratio is u/u_{max} = 0.51. Therefore the flow velocity is u = 0.51 x 34.71 = 17.7 fps. The bore loading is computed for h_e = 3.33 ft and u = 17.7 fps.

Therefore for the 10' wide stairwell wall, $F_d = \frac{3}{4} \times 2.2 \times 1.0 \times 2.0 \times 10(3.33 \times 17.7^2)/1000 = 34.4\ kips$

These loads are applied to the walls as a uniformly distributed pressure of 34.4/(10x3.33) = 1,033 psf over the lower 3.33 ft of the walls. This will not govern when compared with the pressure from hydrodynamic drag in Eqn. 6.10-5a.

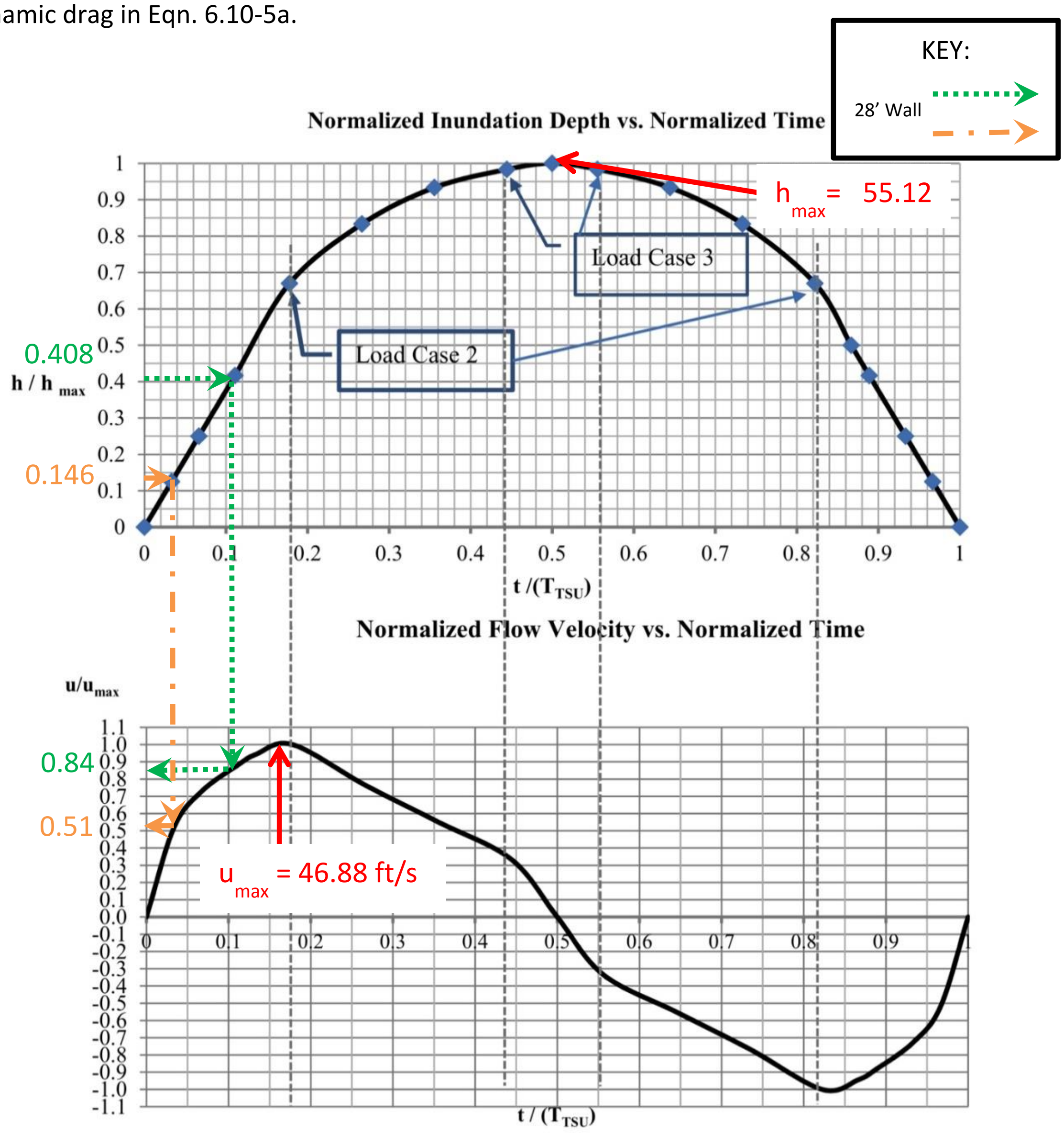


Figure D-45: Determining "u" for Eqn. 6.10-5b with ASCE 7-16 Figure 6.8-1

D.13.3 Hydrodynamic Pressures associated with Slabs – Section 6.10.3

D.13.3.1 Flow Stagnation Pressure – Section 6.10.3.1

The Mechanical/Electrical room on Gridline D between Gridlines 5 and 6 is enclosed on all sides by structural walls. Tsunami flow entering through the two door openings will result in flow stagnation pressurization of this room, given by **Eqn. 6.10-8** as:

$$P_p = \frac{1}{2}\rho_s I_{tsu} u^2$$

Assuming that the door openings are 7 ft high, the stagnation pressurization is based on the maximum flow velocity occurring at this or greater depths, ie. when the door opening is fully submerged. The flow velocity will therefore be the maximum of 34.71 fps which occurs when the flow depth is 15.25 ft (**Figure 6.8-1**, LC2). Therefore;

$$P_p = \frac{1}{2} \times 2.2 \times 1.0 \times 34.71^2 = 1{,}325\ psf$$

The structural walls surrounding this room must be evaluated for an outward pressure of 1,325 psf, in combination with gravity loads per **Section 6.8.3.3**. The floor slab above this room must be designed for a net uplift pressure given by $0.9D + F_{TSU}$ = -0.9x100 + 1,325 = 1,235 psf upwards. This will require additional top reinforcement in this slab and shear reinforcement around the slab perimeter. In order to reduce the amount of additional reinforcement, one could perform a non-linear analysis of the floor slab following the provisions of ASCE 41. A simpler alternative may be to design the floor slab in the mechanical room as a breakaway slab, as shown in **Figure D-46,** in order to relieve pressure. This will apply to all levels up to h_{max}

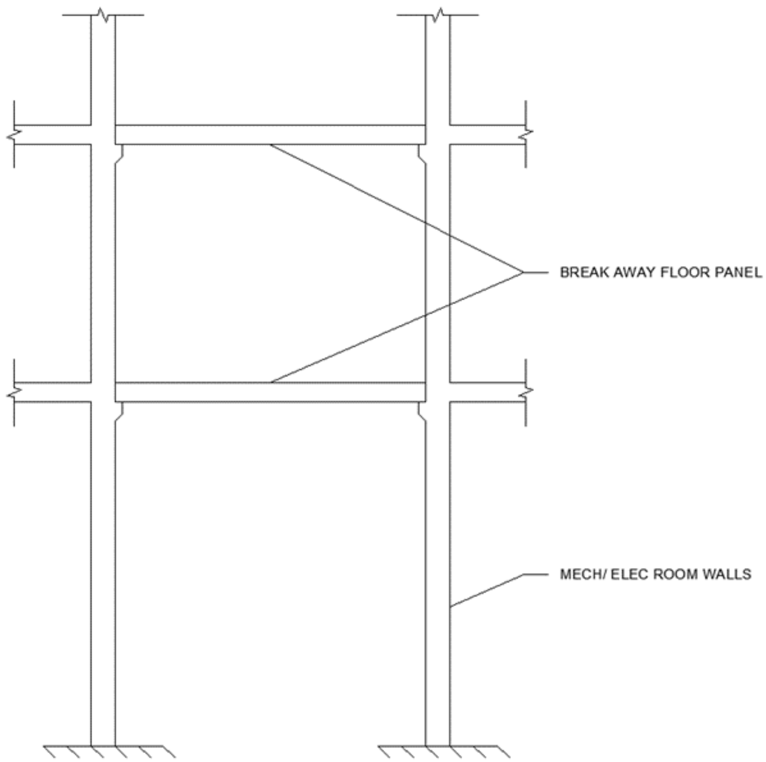


Figure D-46: Mechanical/Electrical room break-away floor panels applied to all levels up to h_{max}

D.13.3.2 Hydrodynamic Surge Uplift at Horizontal Slabs – Section 6.10.3.2

If slabs are submerged during the tsunami, they must be designed for uplift, with a specified minimum of 20 psf (**Section 6.10.3.2.1**). The uplift may increase if the ground floor is sloped, causing an upward component of flow velocity (**Section 6.10.3.2.2**). This is not the case for this building.

The resulting minimum uplift of 20 psf is much smaller than the dead weight of the slab (100 psf), therefore this uplift will not affect the slab design.

D.13.3.3 Tsunami Bore Flow Entrapped in Structural Wall-Slab Recesses – Section 6.10.3.3

If a tsunami bore is entrapped in a structural wall-slab recess, then large pressures can develop on the slab and wall (**Section 6.10.3.3.1**). Although tsunami bores are anticipated at this location, the flow can pass freely around the wall elements in this building. Therefor this condition does not apply.

D.14 Debris Impact Loads - Section 6.11

The inundation depth exceeds 3 feet, therefore exterior structural elements below the flow depth must be designed for debris impact loads.

D.14.1 Alternative Simplified Debris Impact Static Load - Section 6.11.1

In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given by **Eqn. 6.11-1**:

$$F_i = 330C_00I_{tsu} = 330 \times 0.65 \times 1.0 = 214.5\ kips$$

Since the building location is not in an impact zone for shipping containers, ships, and barges, this force can be reduced to 50%, or 107.25 kips. This load must be applied to the 20" square exterior columns as a static lateral load at points critical for flexure and shear, in combination with gravity loads on the column. It is not combined with other tsunami loads and it need not be applied to interior columns.

This equivalent static impact load of 107.25 kips must also be applied to any structural walls on the perimeter of the building. This applies to the 28 ft wide elevator walls on both exterior sides of the building (GLs A and D) since impact must be considered during inflow and outflow conditions. Evaluation of the wall capacity is based on a tributary wall width of half the wall height. Since the wall unbraced height is (12' – 8"/12) = 11.33', the tributary width is 5.67 ft.

D.15 Column Design for Tsunami Loads

D.15.1 Typical Exterior Column Design

A typical exterior column is chosen at Grid Intersection A-3 from **Figure D-16**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category C. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure D-47** and **Figure D-48** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the end of the column, where *d* = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "$d + h_e$" from the end of the column, where $d + h_e$ = 17.5 + 20 = 37.5 in.

Floor 1 – 7

End Section (A)

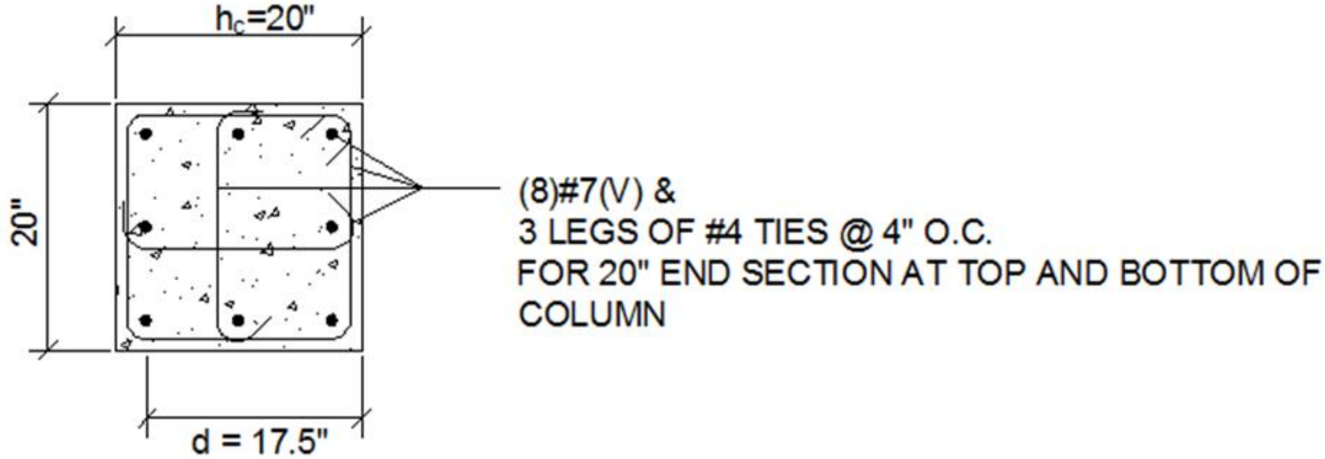


Figure D-47: Exterior column, cross-section at end of column at all floor levels based on SDC D design.

Center Section (B)

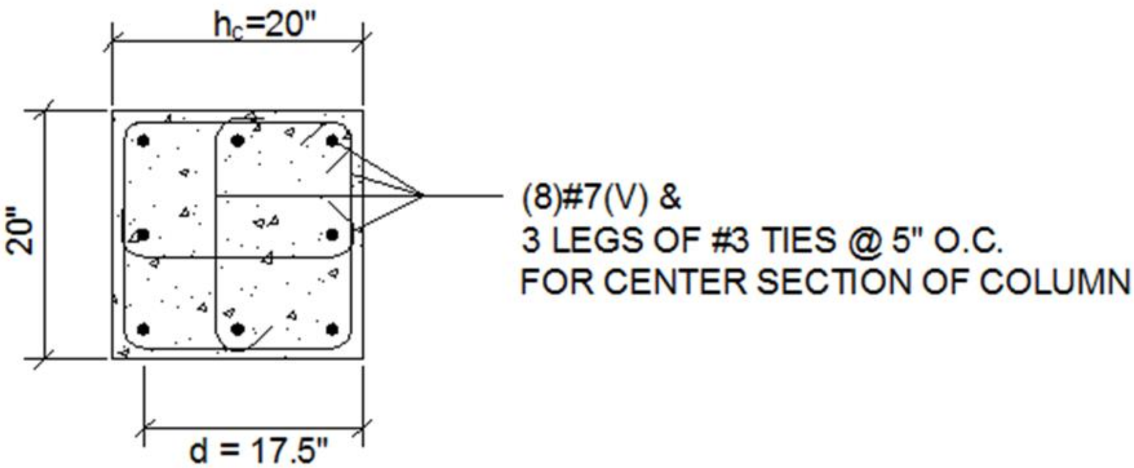


Figure D-48: Exterior column, cross-section at center of column at all floor levels based on SDC D design.

D.15.1.1 Gravity Load Calculation (for completeness)

The gravity load tributary widths are 28 ft and 14.58 ft in the longitudinal and transverse directions respectively. Dead load at base of the column is:

$$P_D = [128(14.58)(28)(6)+ 123(14.58)(28)+ 90(28)(6)+1.67^2(150)(66)]/1000 = 406\ k$$

Floor Live load reduction factor = $0.25+15/[4(14.58)(28)(5)]^{0.5} = 0.402$*, therefore, column base live load is:*

$$P_L = 0.402[55(14.58)(28)(6)]/1000 = 54.2\ k$$

Roof Live Load reduction factor = $R_1R_2= [1.2-(0.001)(14.58)(28)](1.0) = 0.792$*, therefore, roof live load is:*

$P_{Lr} = 0.792(20)(14.58)(28)/1000 = 6.47kF$

Hydrodynamic loads from Load Case 2 will govern over LC1 and LC3 for this column. Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

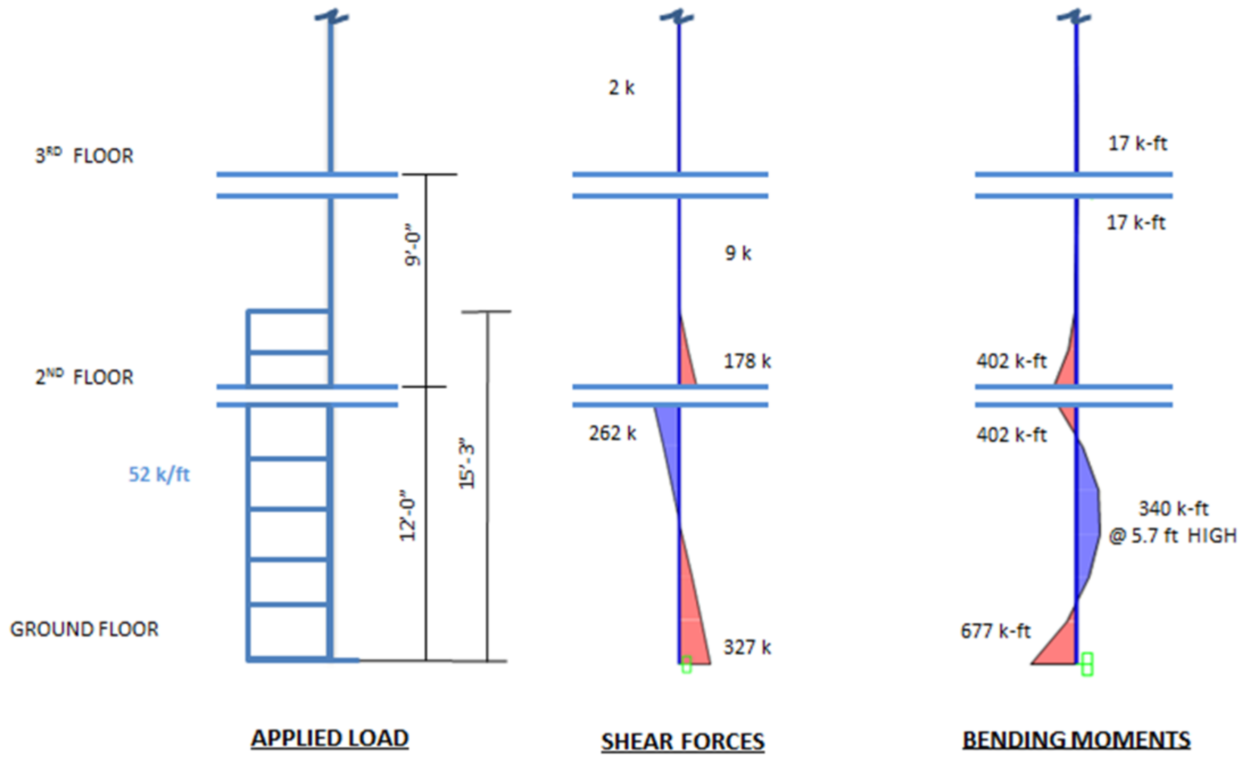


Figure D-49: Hydrodynamic loading on exterior column of Waikīkī residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike is assumed to act just below each inundated floor slab for the maximum shear and near the mid-height of the clear column height for maximum bending moments. Samples of the resulting shear force and bending moment diagrams are provided below. Similar diagrams and similar shear and bending moments would result if the impact load was applied at the other end of each column.

Impact load at d:

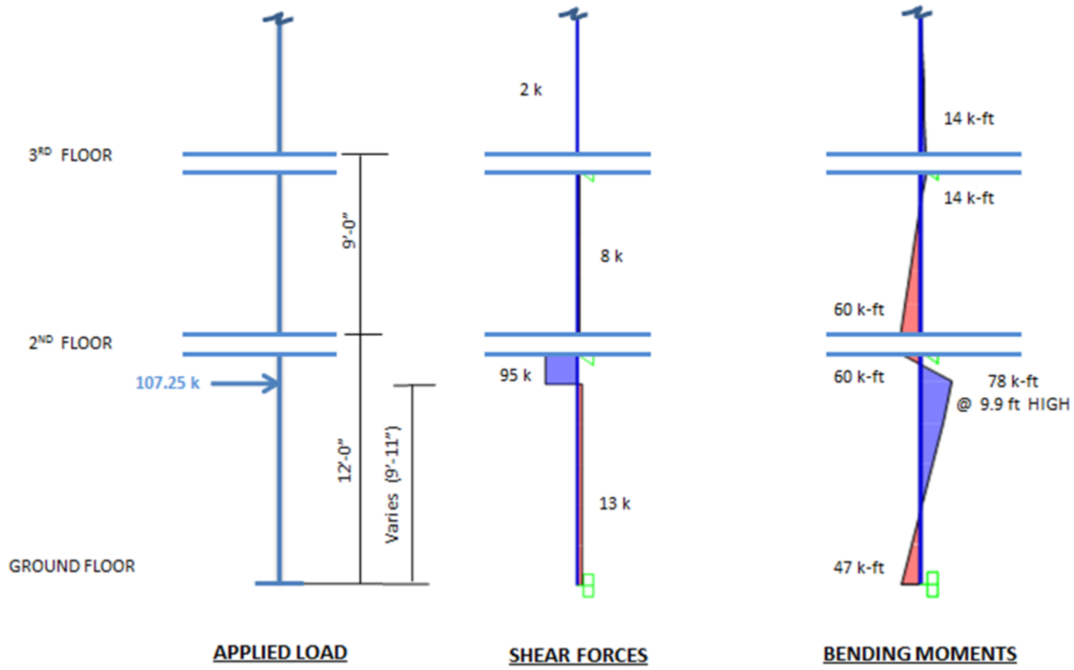


Figure D-50: Impact load applied at "d" away from the end of column on the ground floor

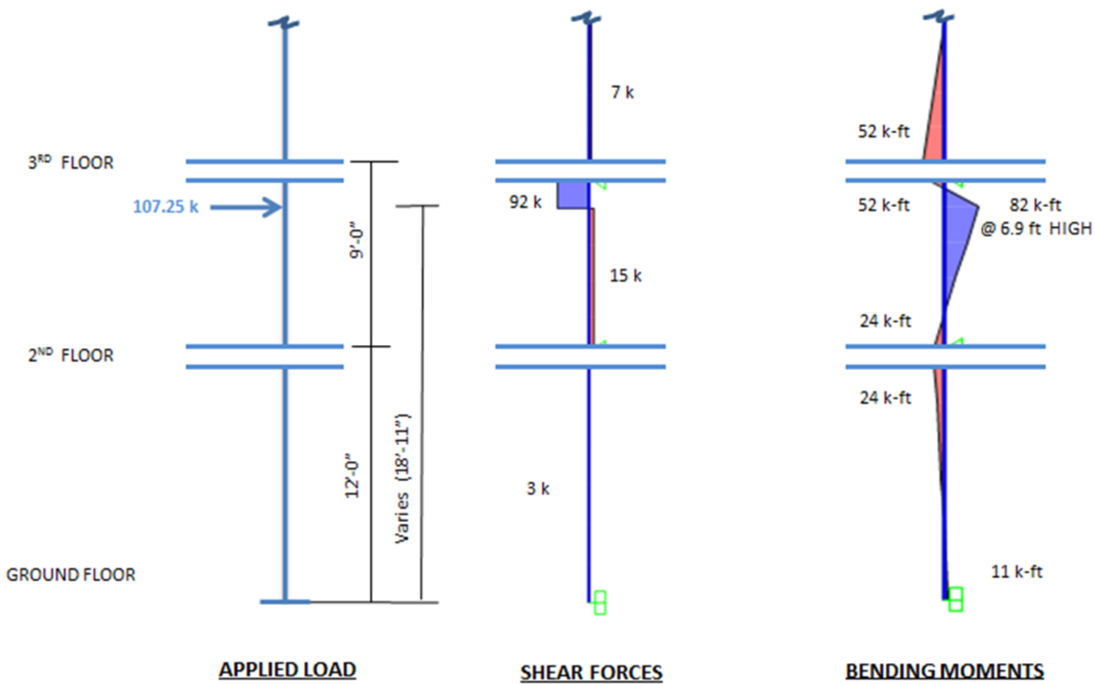


Figure D-51: Impact load applied at "d" away from the end of column on the 2nd floor

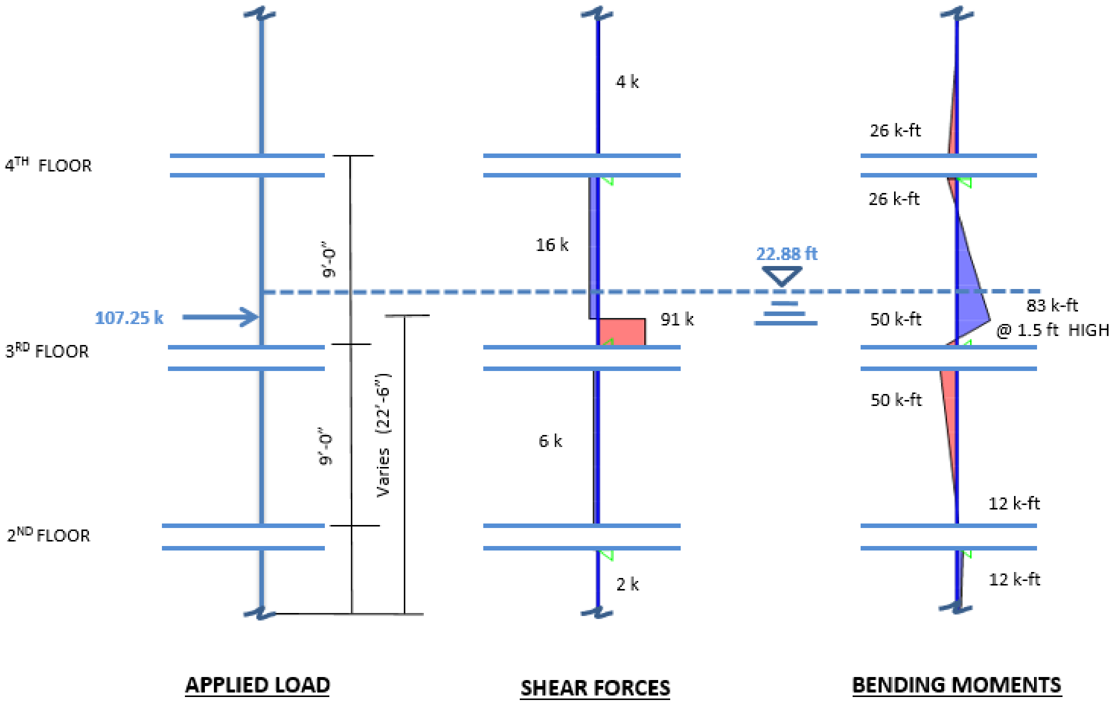


Figure D-52: Impact load applied at "d" away from the end of column on the 3rd floor

Impact load at d + h_c:

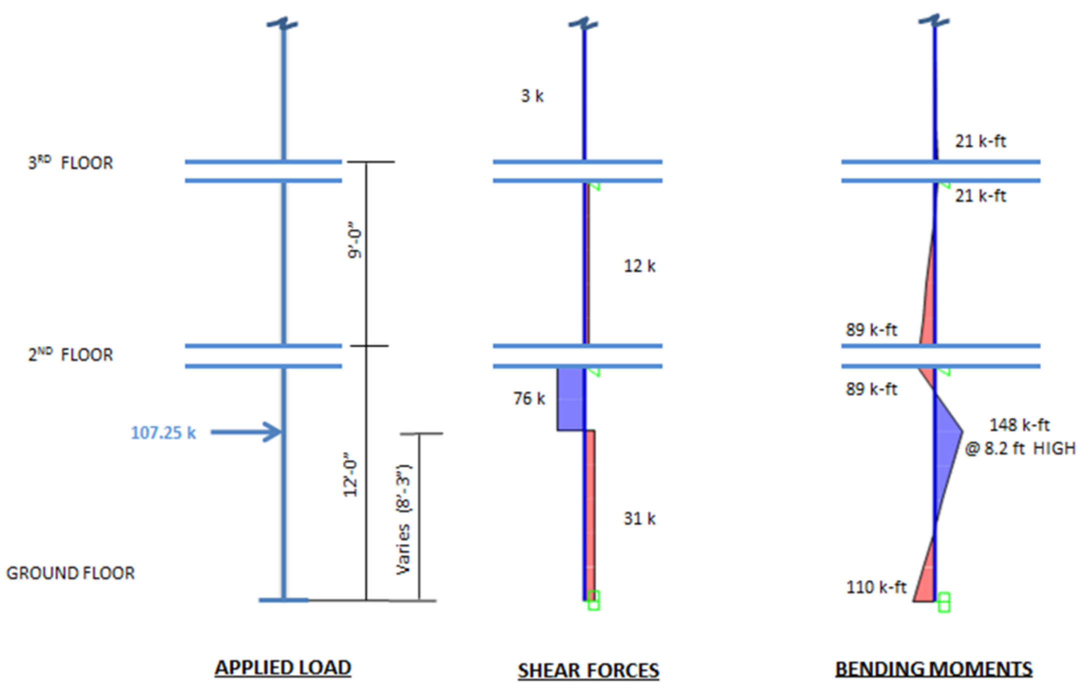


Figure D-53: Impact load applied at "d + h_c" away from the end of column on the ground floor

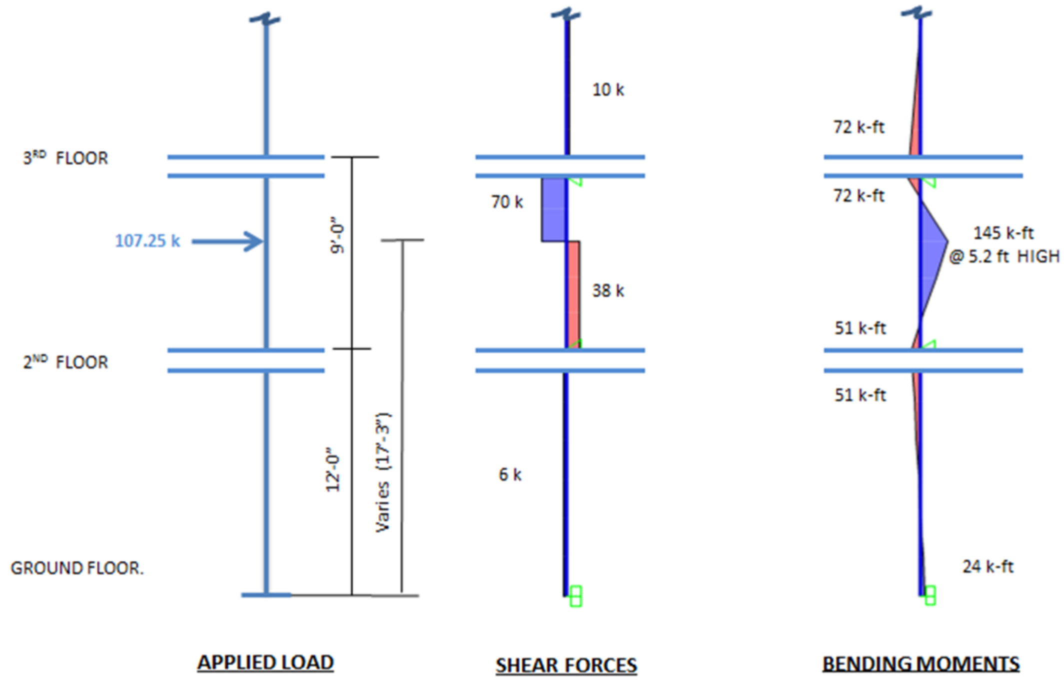


Figure D-54: Impact load applied at "d + h_c" away from the end of column on the 2nd floor

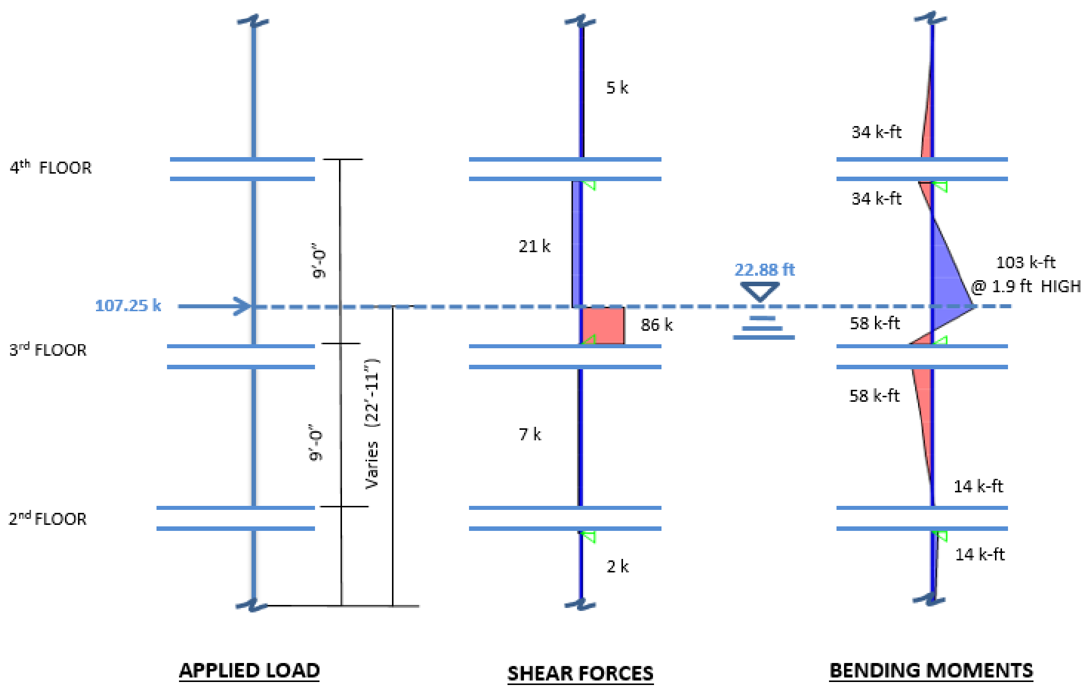


Figure D-55: Impact load applied at 1.9 ft away instead of "d + h_c" away from the end of column on the 3rd floor

Impact load at mid-height:

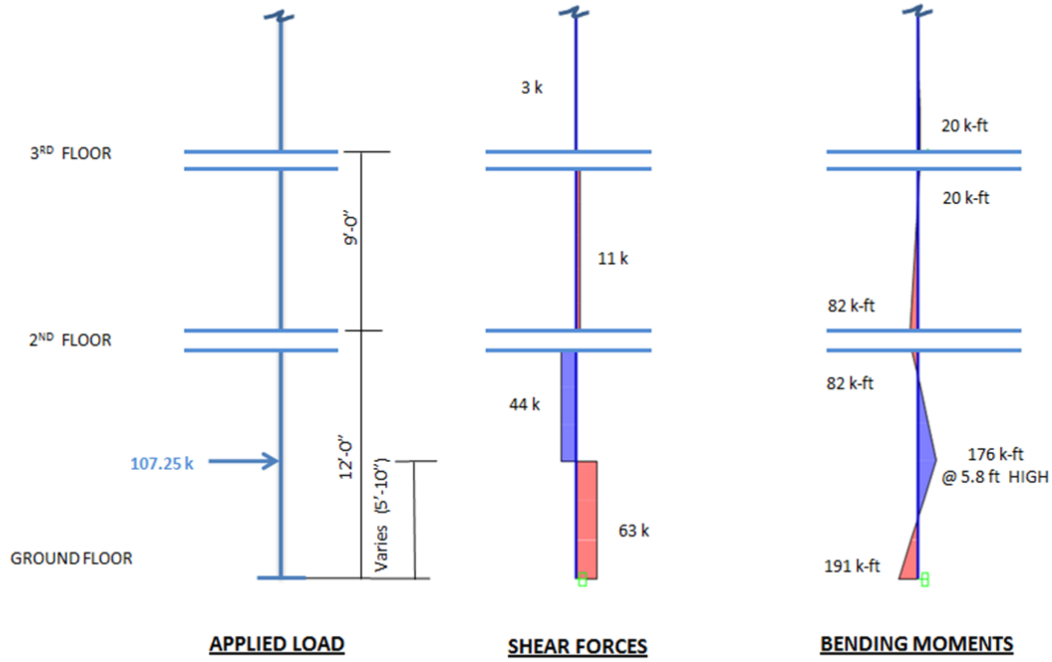


Figure D-56: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

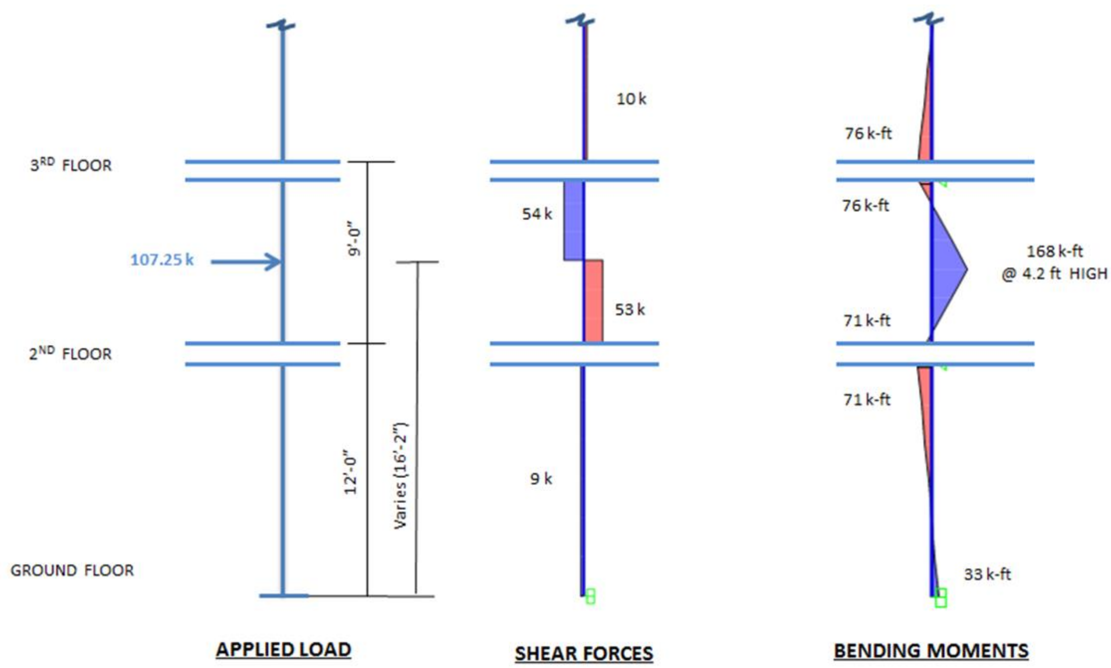


Figure D-57: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

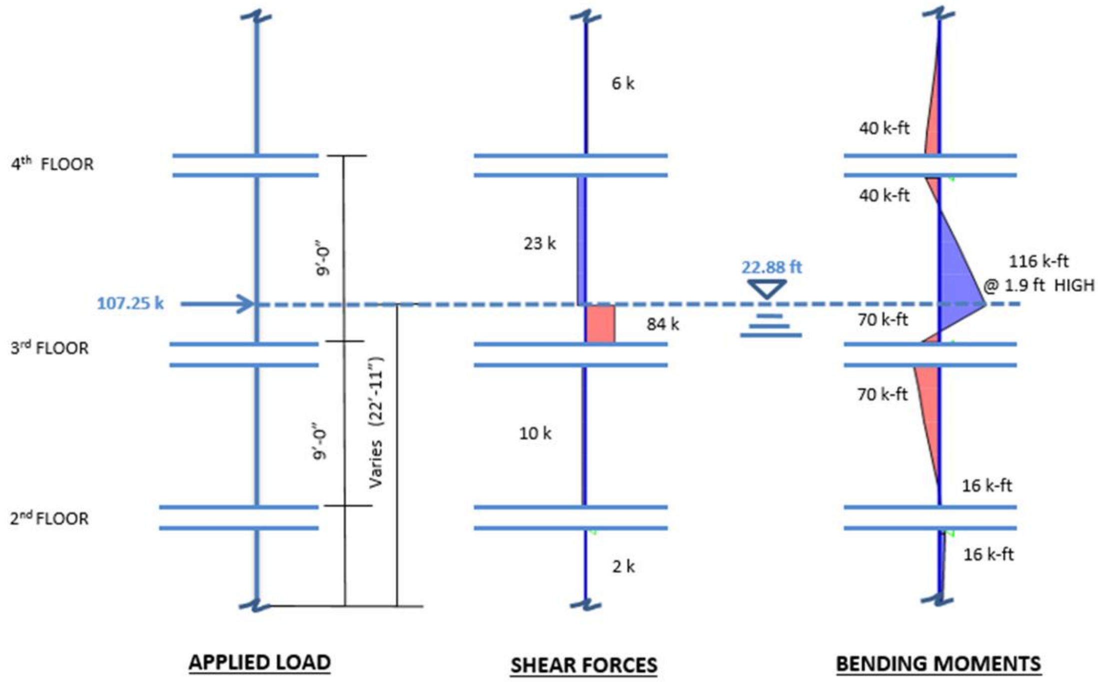


Figure D-58: Impact load applied at 1.9 ft away instead of mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

Table D-6 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** both for hydrodynamic drag (Hydro) and long impact (Impact). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table D-6: Results from loading conditions of Waikīkī residential building exterior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
677	514.3	251	164	1.2D+Ftsu+0.5L (Hydro)
677	365.4	251	164	0.9D+Ftsu (Hydro)
191	514.3	95	76	1.2D+Ftsu+0.5L (Impact)
191	365.4	95	76	0.9D+Ftsu (Impact)
Floor 2				
402	440.8	103	20	1.2D+Ftsu+0.5L (Hydro)
402	313.2	103	20	0.9D+Ftsu (Hydro)
168	440.8	92	70	1.2D+Ftsu+0.5L (Impact)
168	313.2	92	70	0.9D+Ftsu (Impact)
Floor 3				
17	367.4	2	2	1.2D+Ftsu+0.5L (Hydro)
17	261	2	2	0.9D+Ftsu (Hydro)
116	367.4	91	21	1.2D+Ftsu+0.5L (Impact)
116	261	91	21	0.9D+Ftsu (Impact)
Floor 4				
4	293.9	1	1	1.2D+Ftsu+0.5L (Hydro)
4	208.8	1	1	0.9D+Ftsu (Hydro)
40	293.9	5	5	1.2D+Ftsu+0.5L (Impact)
40	208.8	5	5	0.9D+Ftsu (Impact)
Floor 5				
1	220.4	0	0	1.2D+Ftsu+0.5L (Hydro)
1	156.6	0	0	0.9D+Ftsu (Hydro)
10	220.4	3	3	1.2D+Ftsu+0.5L (Impact)
10	156.6	3	3	0.9D+Ftsu (Impact)
Floor 6				
0	146.9	0	0	1.2D+Ftsu+0.5L (Hydro)
0	104.4	0	0	0.9D+Ftsu (Hydro)
2	146.9	1	1	1.2D+Ftsu+0.5L (Impact)
2	104.4	1	1	0.9D+Ftsu (Impact)
Floor 7				
0	73.5	0	0	1.2D+Ftsu+0.5L (Hydro)
0	52.2	0	0	0.9D+Ftsu (Hydro)
2	73.5	0	0	1.2D+Ftsu+0.5L (Impact)
2	52.2	0	0	0.9D+Ftsu (Impact)

D.15.1.2 Combined Gravity and Tsunami Loads

The column chosen at Grid Intersection A-3 from **Figure D-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure D-59 to **Figure D-61** shows the interaction diagram for the typical exterior column including the tsunami load combinations.

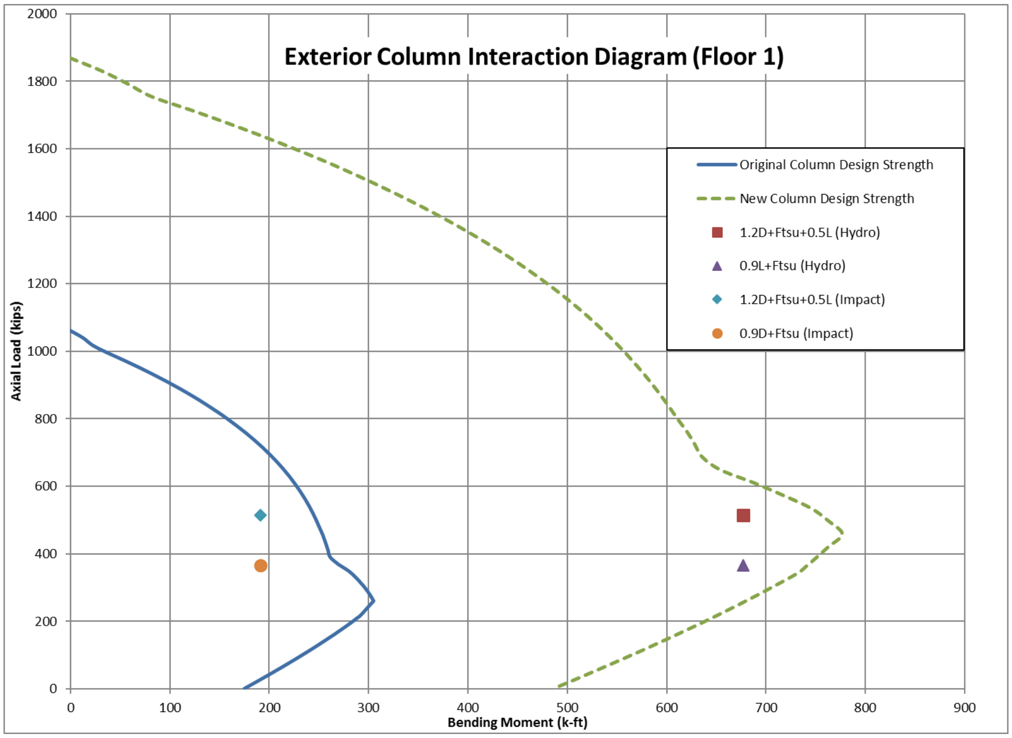


Figure D-59: Interaction diagram for typical ground floor exterior column showing tsunami load combinations

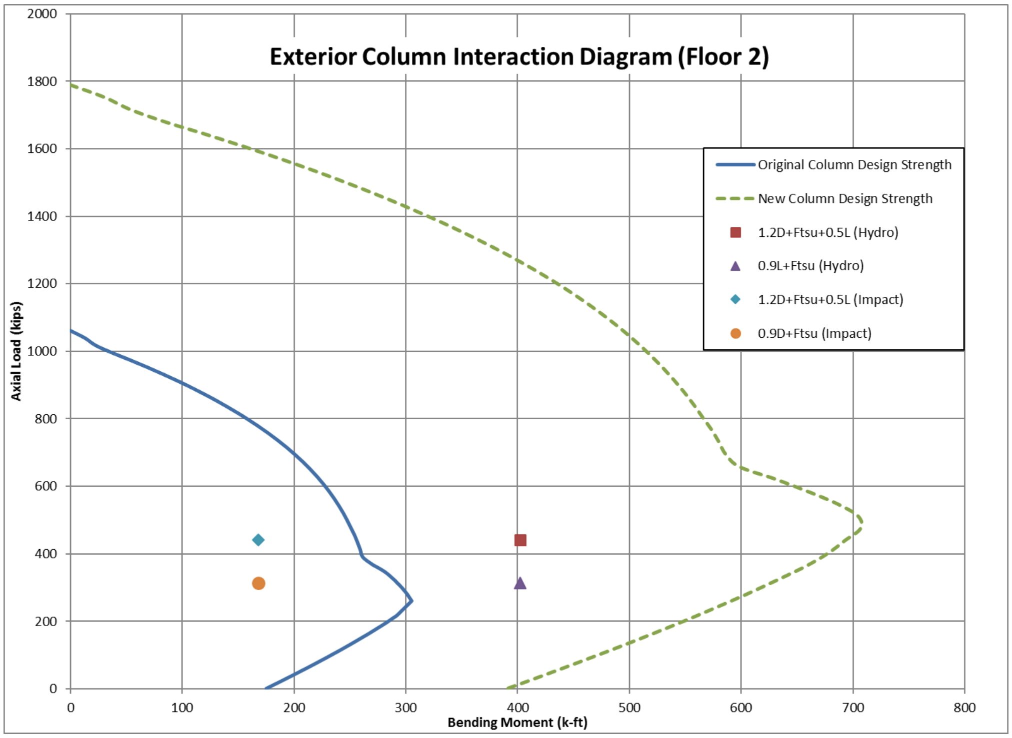


Figure D-60: Interaction diagram for typical 2nd floor exterior column showing tsunami load combinations

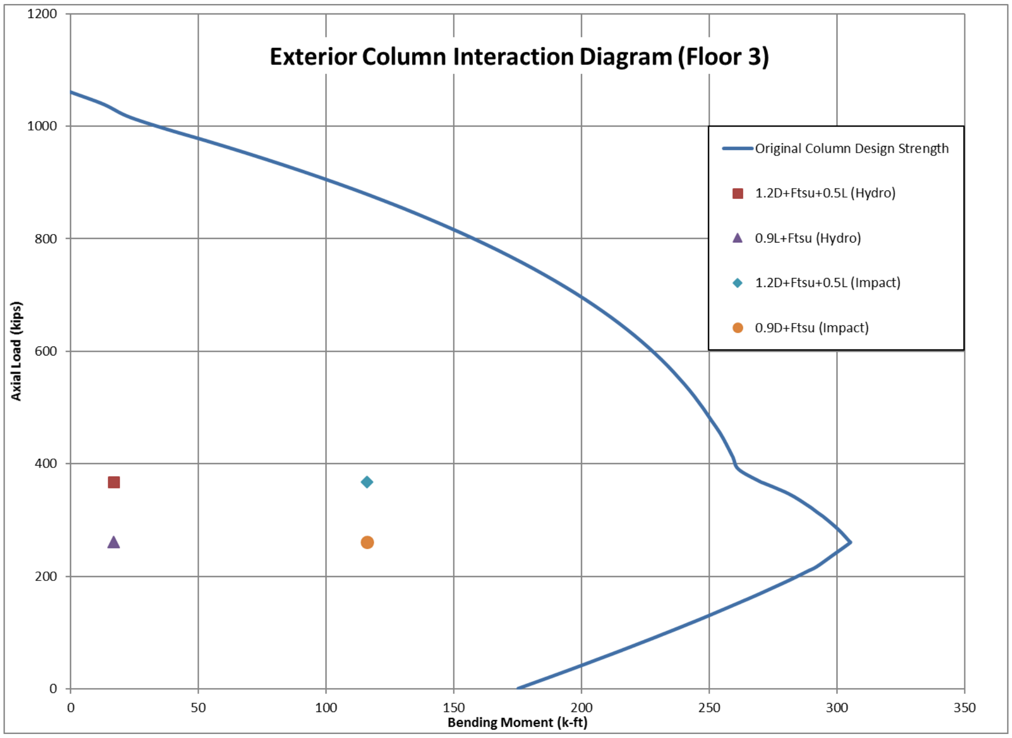


Figure D-61: Interaction diagram for typical 3rd floor exterior column showing tsunami load combinations

By inspection the remaining columns are adequate to resist the tsunami bending moments.

D.15.1.3 New Typical Exterior Column Design

Based on the interaction diagrams shown in **Figure D-59** to **Figure D-61** the original exterior columns are adequate for log impact load, but the columns at the ground and 2nd floors must be strengthened to resist bending due to the hydrodynamic loads. Revised columns designs were developed to satisfy the hydrodynamic loads as shown in in **Figure D-62** to **Figure D-65**. The interaction diagrams for these new columns are shown in **Figure D-59** to **Figure D-60**.

Floor 1

End Section (A)

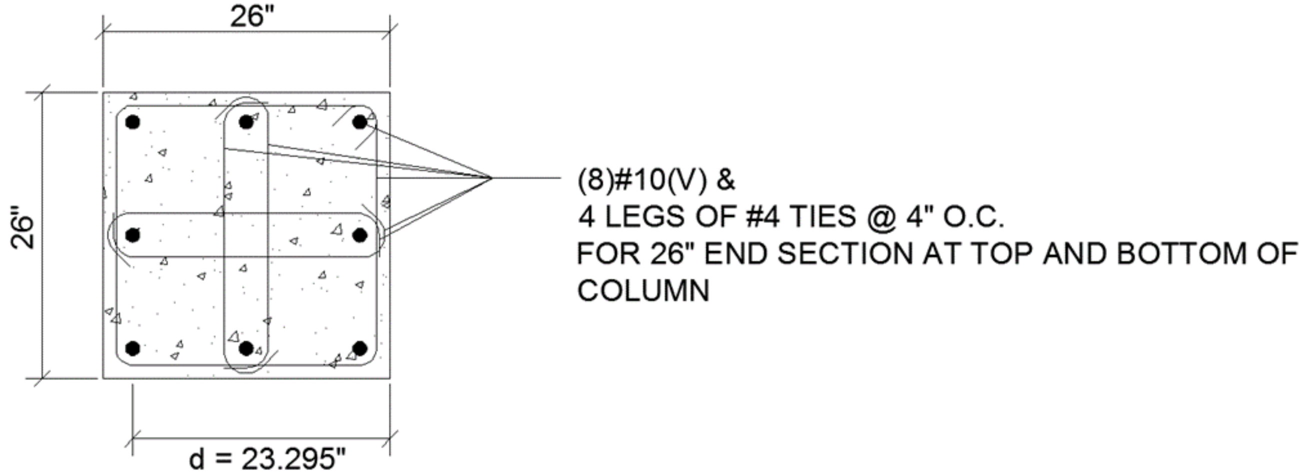


Figure D-62: Exterior column, cross-section at end of column at ground floor level based on tsunami design requirements.

Center Section (B)

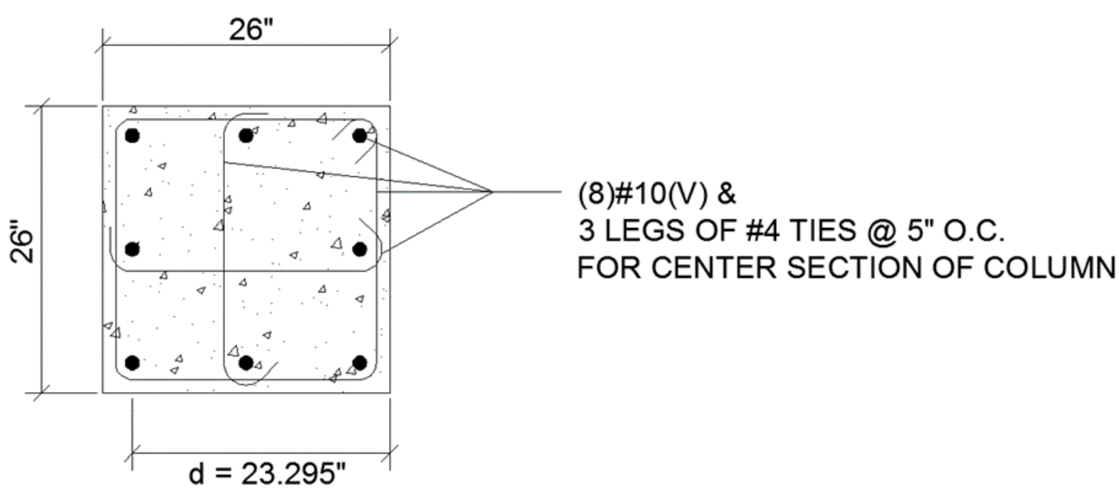


Figure D-63: Exterior column, cross-section at center of column at ground floor level based on tsunami design requirements.

Floor 2

End Section (A)

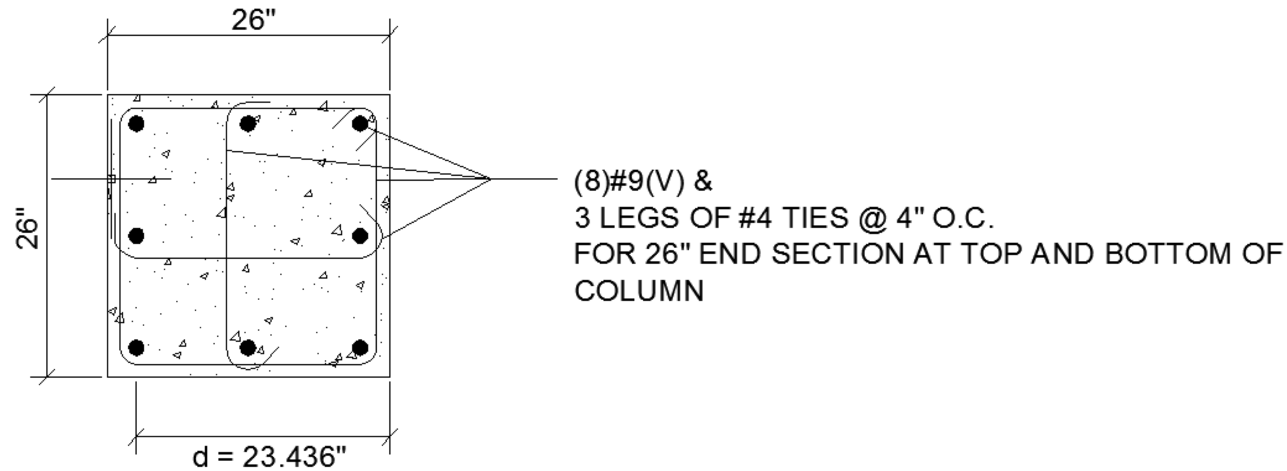


Figure D-64: Exterior column, cross section at end of column at the 2nd floor level based on tsunami design requirements.

Center Section (B)

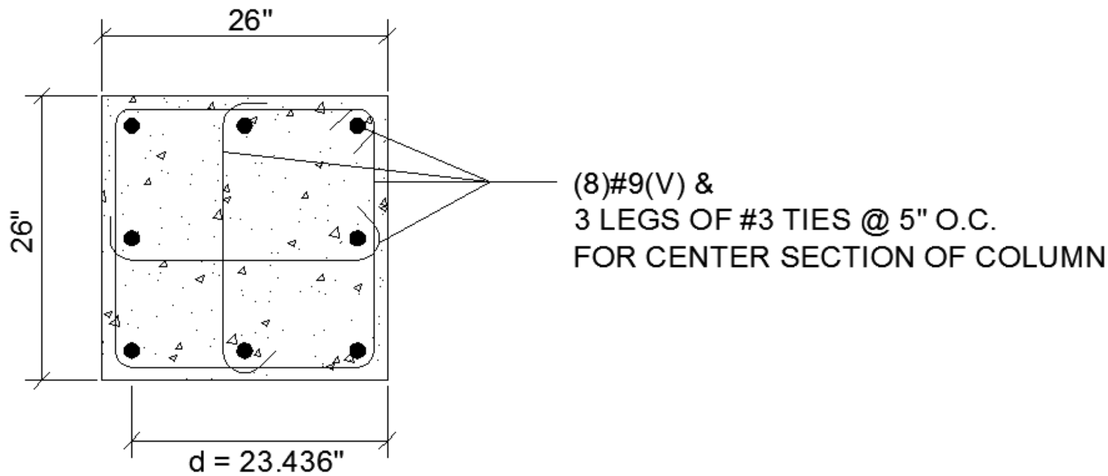


Figure D-65: Exterior column, cross-section at center of column at the 2nd floor level based on tsunami design requirements.

D.15.1.4 Exterior Column Shear Design

Critical Shears in Columns at 1st Floor:

At the critical axial load combination of (0.9D + F_{TSU}) per **Eqn. 6.8.3.3.-1b**, P_u = 365.4 kips.

The shear capacities of the 30”x30” columns with 4 leg #5 Stirrups at 4” o.c. in the end section and 4 leg #4 Stirrups at 4” o.c. in the center section are given by:

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{365{,}400}{2{,}000\times26\times26}\right)26 \times 23.365/1{,}000 = 98$ kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.2)\times60{,}000\times23.365}{4\times1{,}000} = 280$ kips

$V_S = \frac{A_v f_y d}{s} = 280 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 26 \times 23.365 = 307 \text{ kips} \therefore \text{use } 280 \text{ kips}$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(4\times0.11)\times60{,}000\times23.365}{5\times1{,}000} = 123$ kips.

$V_S = \frac{A_v f_y d}{s} = 123 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 26 \times 23.365 = 307 \text{ kips} \therefore \text{use } 123 \text{ kips}$

Therefore in the end sections, $\text{Ø}V_n = 0.75\ (98 + 280) = 283$ k

and in the center sections, $\text{Ø}V_n = 0.75\ (98 + 123) = 166$ k.

At d: $V_u = 251$ k < $\phi V_n = 283$ k, therefore the column is adequate for shear at the end sections.

At $d + h_c$: $V_u = 164$ k < $\phi V_n = 166$ k, therefore the column is adequate for shear at the center section

Critical Shears in Columns at 2nd Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, $P_u = 313.2$ kips.

The shear capacities of the 26"x26" columns with 3 leg #4 Stirrups at 4" o.c. in the end section and 3 leg #3 Stirrups at 5" o.c. in the center section are given by:

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{313{,}200}{2{,}000\times26\times26}\right)26 \times 23.436/1{,}000 = 95$ kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times23.436}{4\times1{,}000} = 211$ kips

$V_S = \frac{A_v f_y d}{s} = 211 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 26 \times 23.436 = 308 \text{ kips} \therefore \text{use } 211 \text{ kips}$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times23.436}{5\times1{,}000} = 93$ kips.

$V_S = \frac{A_v f_y d}{s} = 93 \text{ kips} \ngtr 8\sqrt{f_C'}\text{ b d} = 8 \times \sqrt{4{,}000} \times 26 \times 23.436 = 308 \text{ kips} \therefore \text{use } 93 \text{ kips}$

Therefore in the end sections, $\text{Ø}V_n = 0.75\ (95 + 211) = 229$ k

and in the center sections, $\text{Ø}V_n = 0.75\ (95 + 93) = 141$ k.

At d: V_u = 103 k < ϕV_n = 229 k, therefore the column is adequate for shear at the end sections.

At d + h_c*:* V_u = 70 k < ϕV_n = 141 k, therefore the column is adequate for shear at the center section

Critical Shears in Columns at 3rd Floor:

At the critical axial load combination of ($0.9D + F_{TSU}$) per **Eqn. 6.8.3.3.-1b**, P_u = 261 kips.

The shear capacities of the 20"x20" columns with 3 leg #4 Stirrups at 4" o.c. in the end section and 3 leg #3 Stirrups at 5" o.c. in the center section are given by:

$$\emptyset V_n = \emptyset(V_C + V_S)$$

where $V_C = 2\sqrt{f_C{}'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{261{,}000}{2{,}000\times20\times20}\right)20 \times 17.5625/1{,}000$ = 59 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.2)\times60{,}000\times17.5625}{4\times1{,}000}$ = 158 kips

$$V_S = \frac{A_v f_y d}{s} = 158 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\mathrm{b\ d} = 8 \times \sqrt{4{,}000} \times 20 \times 17.5625 = 178 \text{ kips} \therefore \text{use } 158 \text{ kips}$$

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times0.11)\times60{,}000\times17.5625}{5\times1{,}000}$ = 69 kips.

$$V_S = \frac{A_v f_y d}{s} = 69 \text{ kips} \ngtr 8\sqrt{f_C{}'}\,\mathrm{b\ d} = 8 \times \sqrt{4{,}000} \times 20 \times 17.5625 = 178 \text{ kips} \therefore \text{use } 69 \text{ kips}$$

Therefore in the end sections, $\emptyset V_n$ = 0.75 (59 + 158) = 163 k

and in the center sections, $\emptyset V_n$ = 0.75 (59 + 70) = 96 k

At d: V_u = 91 k < ϕV_n = 163 k, therefore the column is adequate for shear at the end sections.

At d + h_c*:* V_u = 21 k < ϕV_n = 96 k, therefore the column is adequate for shear at the center section

By inspection the remaining columns are adequate to resist the tsunami shear force.

Instead of the equivalent static load analysis performed above, it is permissible to use a non-linear analysis following the provisions of ASCE 41, or to perform a non-linear dynamic analysis of the column subjected to the debris impact strike.

D.15.2 Typical Interior Column Design

A typical interior column is chosen at Grid Intersection B-3 from **Figure D-16**. The column is not part of the lateral force resisting system for seismic loads. It will be subject to the same displacements as the lateral resisting system, therefore needs to be detailed accordingly for Seismic Design Category D. It is assumed to have a fixed base at the foundation; therefore a plastic hinge may form at the base of the column. The remainder of the column will not form a plastic hinge, but the slab connection at the column needs to be detailed appropriately for the seismic deformation. The 20 in square column cross section shown in **Figure D-66** and **Figure D-67** was selected based on gravity load design and punching shear capacity of the floor slabs.

The critical shear force occurs at a distance "*d*" from the ends of the column, where d = 20 - 1.5 - 0.5 - 0.5 = 17.5 in. The critical shear force for the center section of the column occurs at "$d + h_c$" from each end of the column, where $d + h_c$ = 17.5 + 20 = 37.5 in.

Floor 1 – 7

End Section (A)

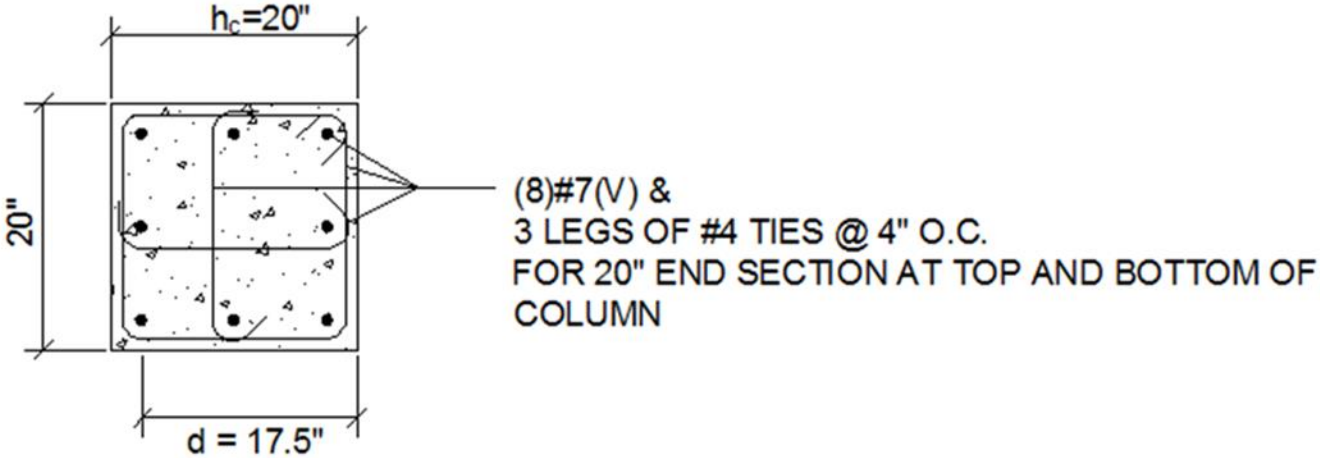


Figure D-66: Interior column, cross-section end of column at all floor levels based on SDC D design.

Center Section (B)

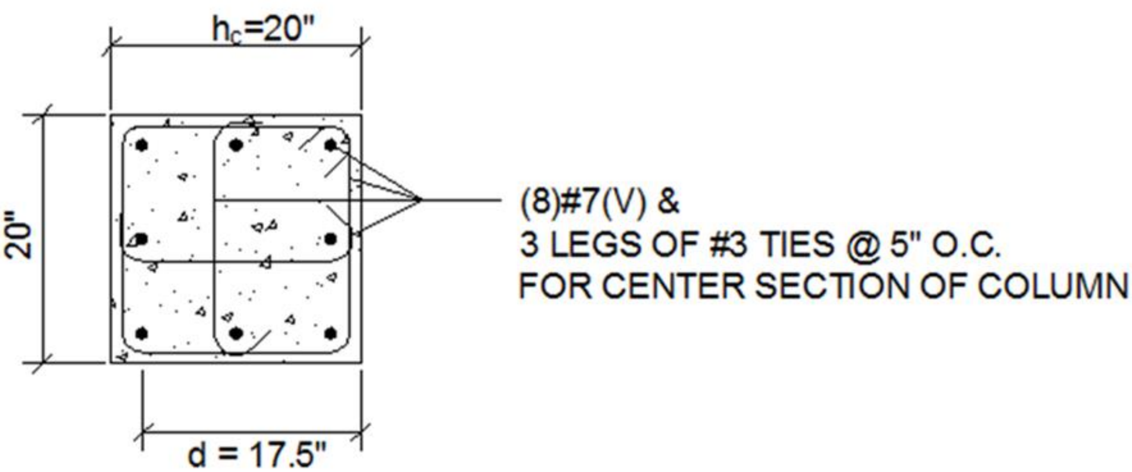


Figure D-67: Interior column, cross-section at center of column at all floor levels based on SDC D design.

D.15.2.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 28 ft and 17.83 ft in the longitudinal and transverse directions respectively. The Dead Load at the base of the column is:

$$P_D = [128(14.58)(28)(6)+ 123(17.83)(28)+1.67^2(150)(66)]/1000 = 472 \text{ k}$$

Floor Live load reduction factor $= 0.25+15/[4(17.83)(28)(6)]^{0.5} = 0.487$*, therefore, column base live load is:*

$$P_L = 0.487[55(17.83)(28)(6)]/1000 = 80.2 \text{ k}$$

Roof Live Load reduction factor $= R_1R_2= [1.2-(0.001)(17.83)(28)](1.0) = 0.701$*, column roof live load is:*

$$P_{Lr} = 0.701(20)(17.83)(28)/1000 = 6.61 \text{ k}$$

Analysis of the column with the applied tsunami loads from Load Case 2 results in the following bending moment and shear force diagrams:

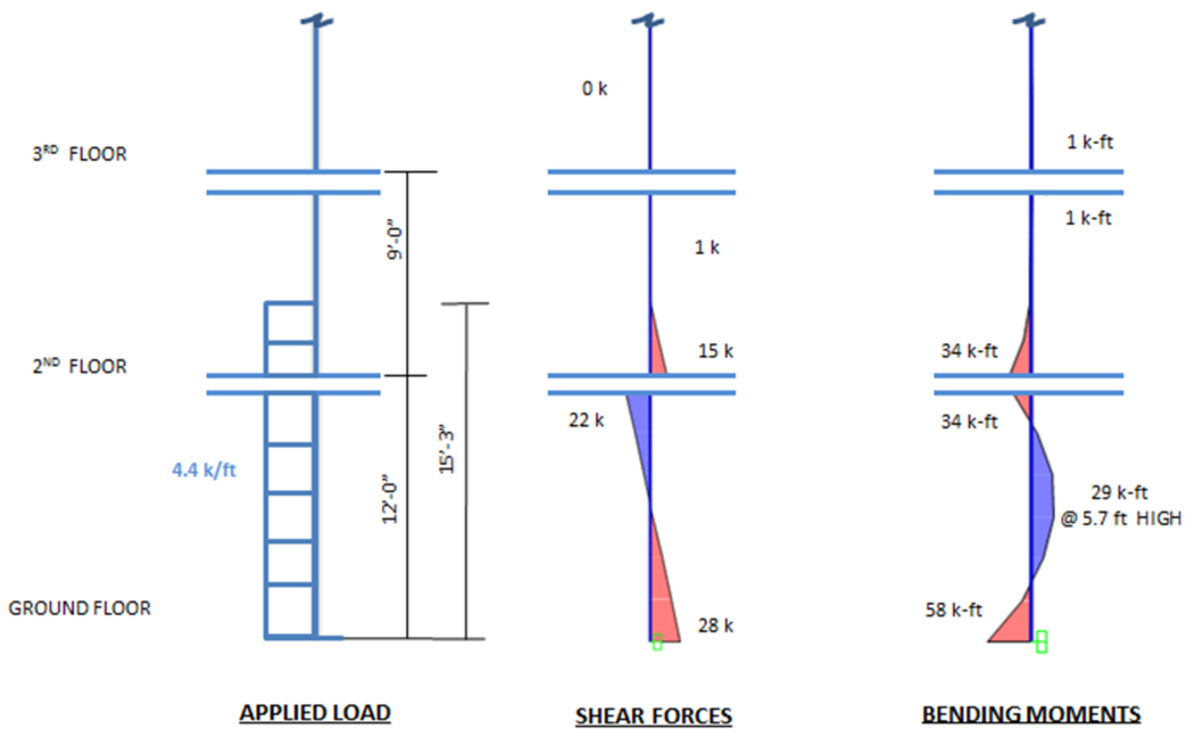


Figure D-68: Hydrodynamic loading on interior column of Waikīkī residential building due to Load Case 2

Table D-7 summarizes the maximum critical load, bending moment and shear forces for all inundated columns using the load combinations provided in **section 6.8.3.3** for the hydrodynamic drag (Hydro). The original column designs will now be evaluated for these load combinations and modified if necessary.

Table D-7: Results from loading conditions of Waikīkī residential building interior column

Moment k-ft	Axial Load Kips	Shear @ d Kips	Shear @ d + h_c Kips	Load Combination
Floor 1				
58	811.8	21	14	1.2D+Ftsu+0.5L (Hydro)
58	566.1	21	14	0.9D+Ftsu (Hydro)
Floor 2				
34	676.5	9	2	1.2D+Ftsu+0.5L (Hydro)
34	471.8	9	2	0.9D+Ftsu (Hydro)
Floor 3				
1	541.2	0	0	1.2D+Ftsu+0.5L (Hydro)
1	377.4	0	0	0.9D+Ftsu (Hydro)
Floor 4				
0	405.9	0	0	1.2D+Ftsu+0.5L (Hydro)
0	283.1	0	0	0.9D+Ftsu (Hydro)
Floor 5				
0	270.6	0	0	1.2D+Ftsu+0.5L (Hydro)
0	188.7	0	0	0.9D+Ftsu (Hydro)
Floor 6				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.4	0	0	0.9D+Ftsu (Hydro)
Floor 7				
0	135.3	0	0	1.2D+Ftsu+0.5L (Hydro)
0	94.4	0	0	0.9D+Ftsu (Hydro)

D.15.2.2 Combined Gravity and Tsunami Loads

The column at Grid intersection B-3 from **Figure D-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**. The column is considered braced against side-sway in the transverse direction.

Figure D-69 shows the interaction diagram for the typical exterior column including the tsunami load combinations.

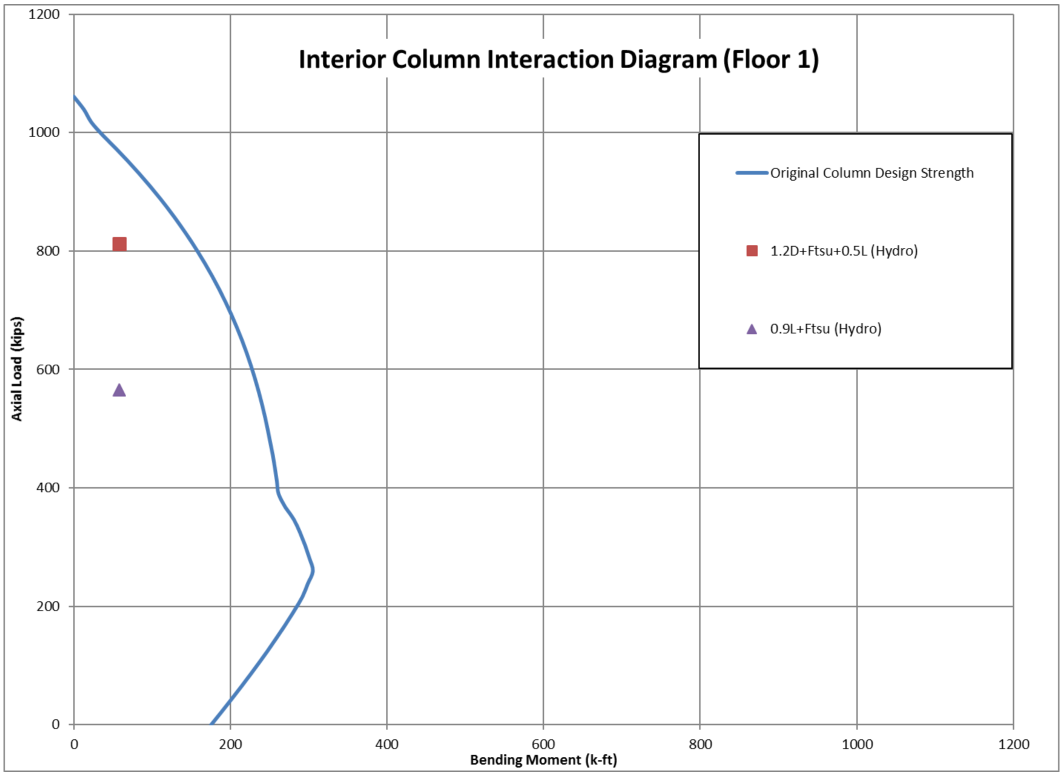


Figure D-69: Interaction diagram for typical ground floor residential interior column showing tsunami load combinations

D.15.2.3 Interior Column Shear Design

Critical Shears in Interior Columns at 1st Floor:

At the critical axial load combination of ($1.2D + F_{TSU} + 0.5L$) per **Eqn. 6.8.3.3.-1a,** P_u = 811.8 kips.

The shear capacities of the existing 20"x20" column with 3 leg #4 Stirrups @ 4" o.c. in the end sections and 3 leg #3 Stirrups @ 5" o.c. in the center section are given by:

$$\text{Ø}V_n = \text{Ø}(V_C + V_S)$$

where $V_C = 2\sqrt{f_C'}\left(1 + \frac{P_u}{2000A_g}\right)bd = 2\sqrt{4000}\left(1 + \frac{811{,}800}{2{,}000\times 20\times 20}\right)20 \times 17.5625/1{,}000$ = 90 kips

and in the end section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.2)\times 60{,}000\times 17.5625}{4\times 1{,}000}$ = 158 kips

and in the center section, $V_S = \frac{A_v f_y d}{s} = \frac{(3\times 0.11)\times 60{,}000\times 17.5625}{5\times 1{,}000}$ = 70 kips

Therefore in the end sections, $\text{Ø}V_n$ = 0.75 (90 + 158) = 186 k

and in the center sections, $\emptyset V_n$ = 0.75 (90 + 70) = 120 k

At d: V_u = 37 k < ϕV_n = 186 k, therefore the column is adequate for shear at the edge.

At d + h_c*:* V_u = 24 k < ϕV_n = 120 k, therefore the column is adequate for shear at the center

By inspection the remaining columns are adequate to resist the tsunami shear force.

D.15.3 Typical Exterior Wall Design

A section of exterior wall along Grid D from **Figure D-16** adjacent to the mechanical room is analyzed. The wall was part of the lateral resisting system for seismic loads, acting as a shear wall for longitudinal forces and boundary element for transverse forces. Seismic Design Category D design and detailing of the 10" thick wall resulted in the reinforcement layout shown in **Figure D-70** to **Figure D-72**. The wall will now be checked for tsunami loads.

For comparative purposes with the debris impact loads, the ultimate shear forces and bending moments are provided for an effective width of wall equal to 5.67 ft. The critical shear force occurs at a distance "*d*" from the base of the wall, where *d* = 10-0.75-1"/2 = 8.75 in.

Floor 1

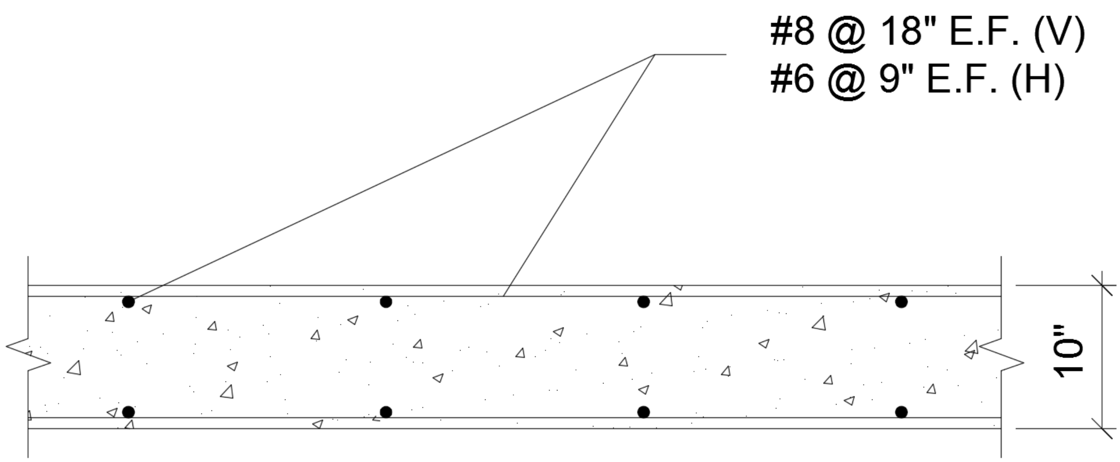


Figure D-70: Segment of exterior wall cross-section at the 1st floor level based on SDC D design.

Floor 2

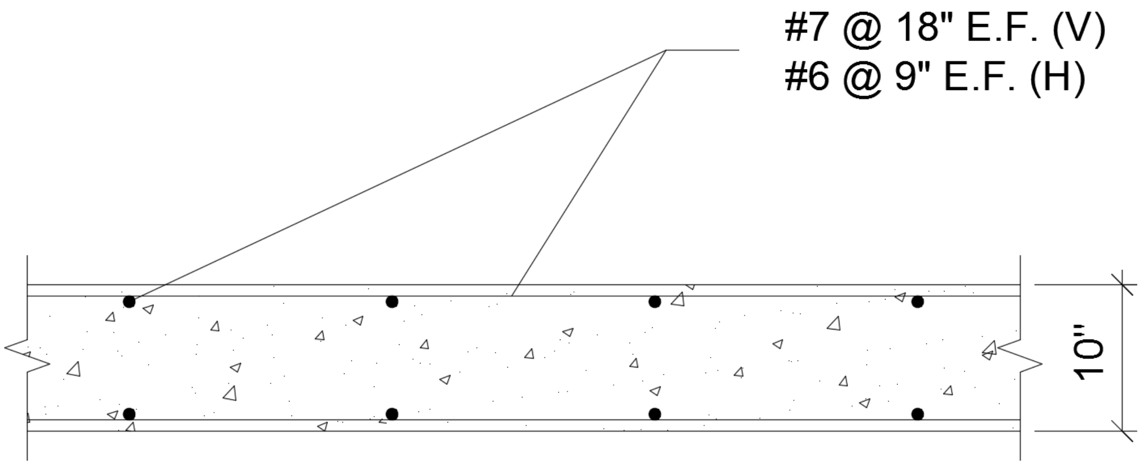


Figure D-71: Segment of exterior wall cross-section at the 2nd floor level based on SDC D design.

Floor 3 – 7

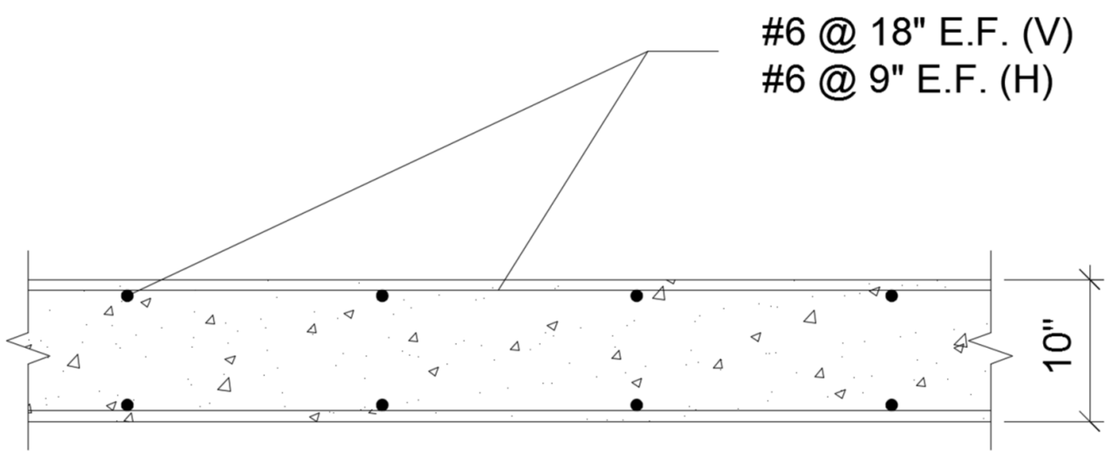


Figure D-72: Segment of exterior wall cross-section at the 3rd – 7th floor level based on SDC D design.

D.15.3.1 Gravity Load Calculation (for completeness)

The gravity load tributary width is 5.5 ft. Gravity loads will be computed per foot width of wall.

The Dead Load at the base of the wall is:

$$P_D = [128(5.5)(1)(6)+123(5.5)(1)+0.83(1)(150)(66)]/1000 = 13.1 \text{ k/ft}$$

Floor Live load reduction: Reduction Factor = $0.25+15/[1(5.5)(28)(6)]^{0.5} = 0.743$

$$P_L = 0.743[55(5.5)(1)(6)]/1000 = 1.35 \text{ k/ft}$$

Roof Live Load reduction: Reduction Factor = $R_1R_2= [1.2-(0.001)(5.5)(28)](1.0) = 1.05$*, Use 1.0*

$P_{Lr} = 20(5.5)(1)/1000 = 0.110$ k/ft

Analysis of a 5.67 foot width of wall with the applied tsunami hydrodynamic loads from Load Case 2 results in the following bending moment and shear force diagrams:

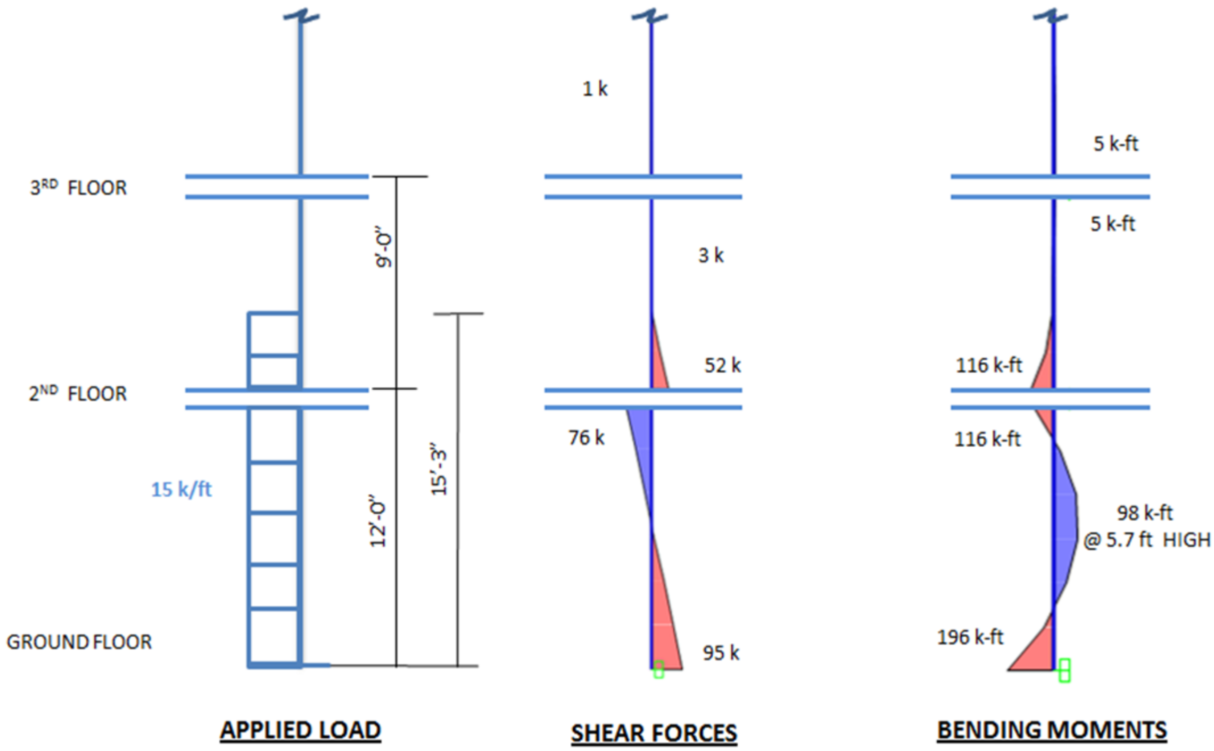


Figure D-73: Hydrodynamic loading on exterior wall of Waikīkī residential building due to Load Case 2

For debris impact loading, the equivalent static load of 107.25 kips resulting from a log strike, acts over an effective width of 5.67 ft, at a point just below the slab at each inundated floor for maximum shear and at the mid-height of the clear column height for maximum bending moments. The resulting shear force and bending moment diagrams for log impact at a distance "d" from the end of the column at each floor level are shown in **Figure D-74** to **Figure D-76**. The resulting shear force and bending moment diagrams for log impact at mid height from the end of the column at each floor level are shown **in Figure D-77** to **Figure D-79**.

Impact load at d:

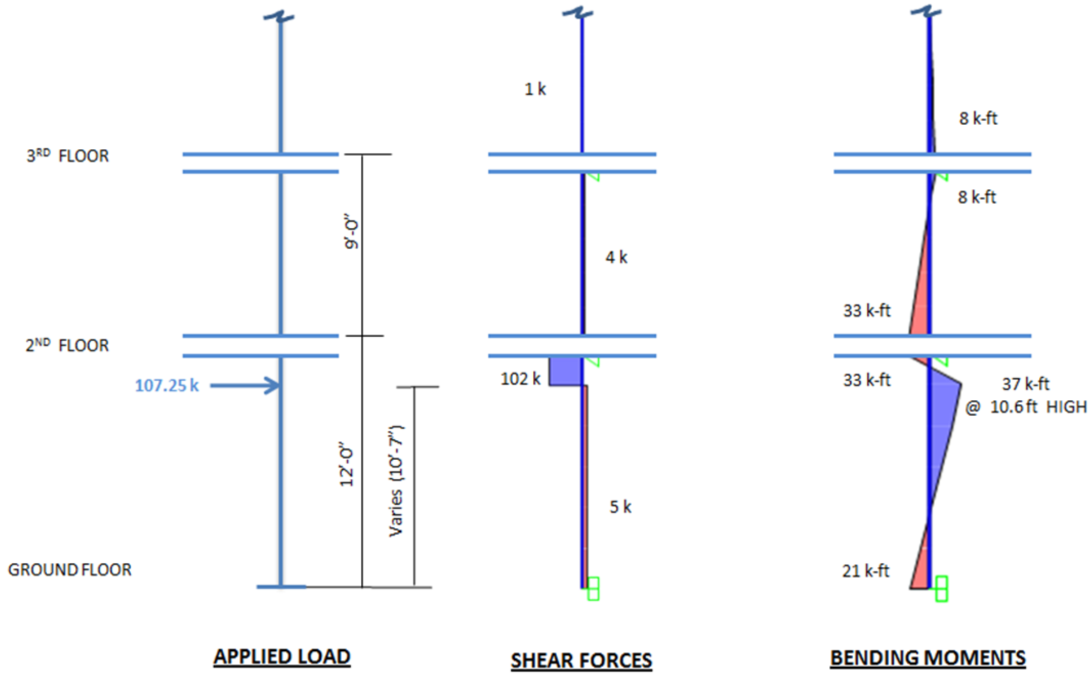


Figure D-74: Impact load applied at d away from the end of column on the ground floor

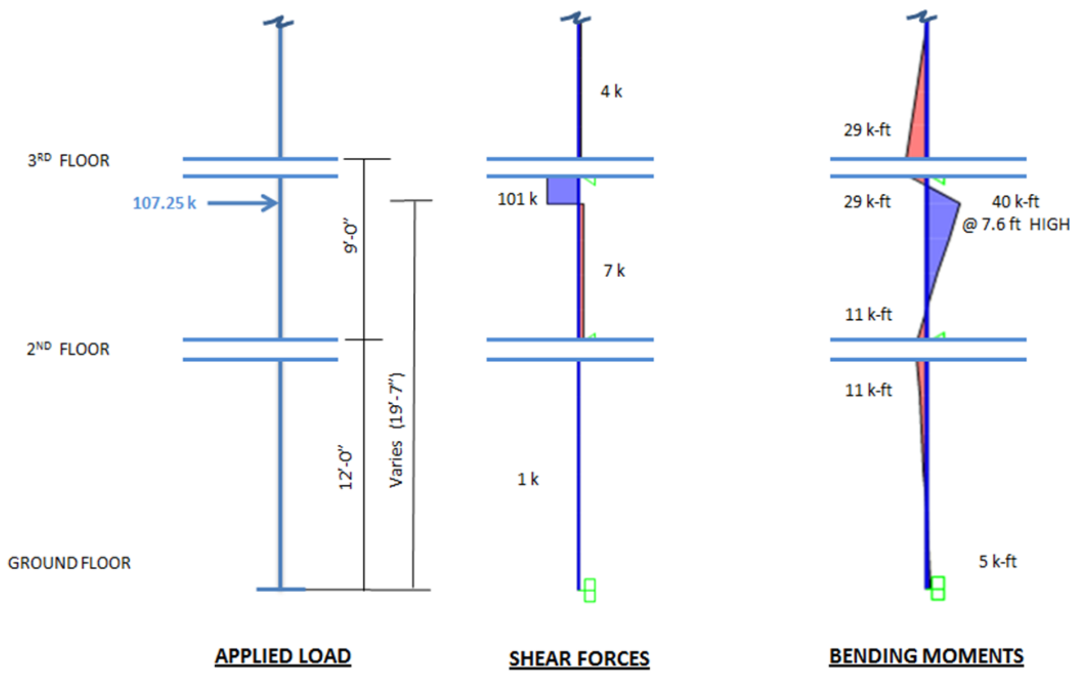


Figure D-75: Impact load applied at d away from the end of column on the 2nd floor

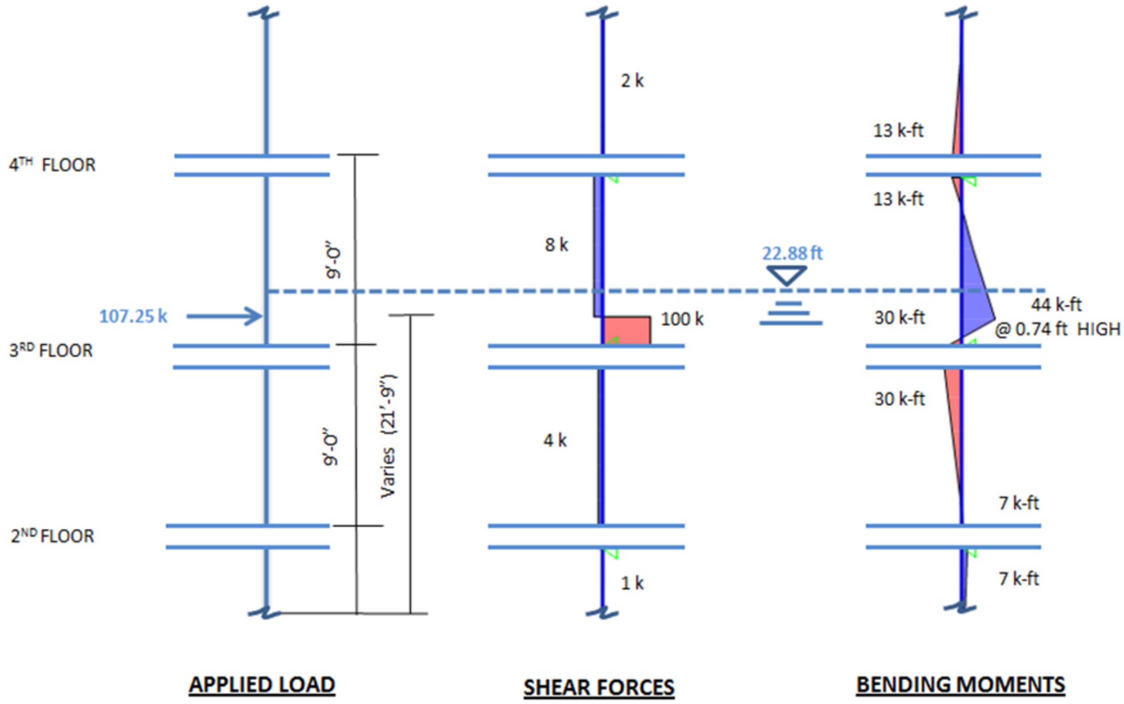


Figure D-76: Impact load applied at d away from the end of column on the 3rd floor

Point load at Mid-heights:

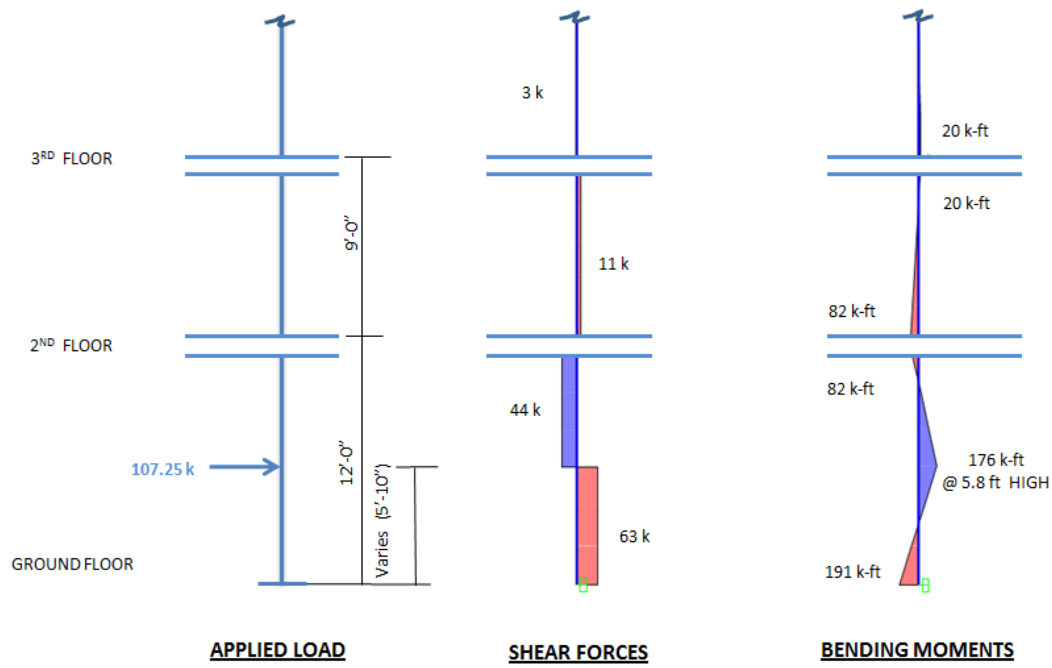


Figure D-77: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the ground floor column

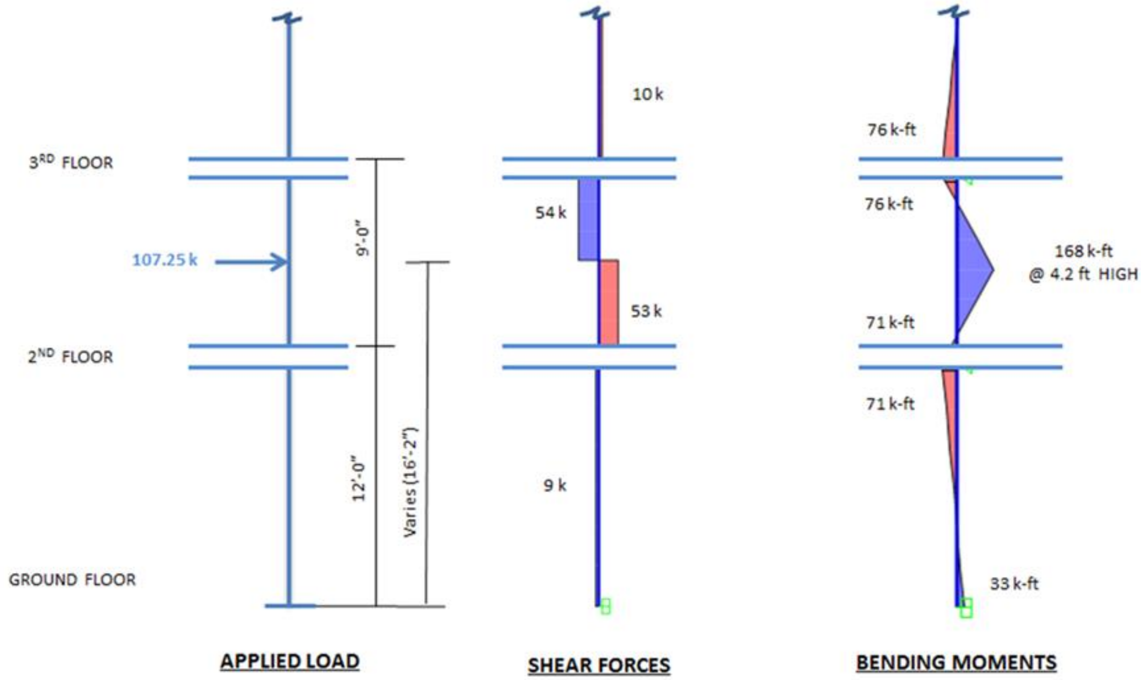


Figure D-78: Impact load applied at mid-height of assumed lateral restraint points at top and bottom of the 2nd floor column

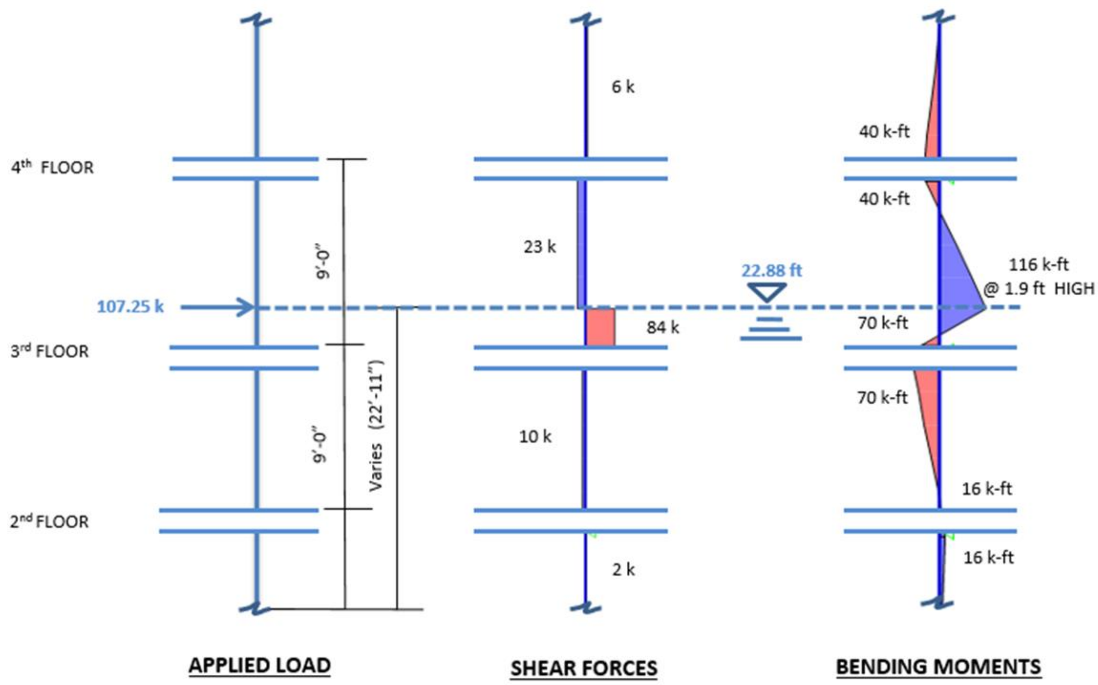


Figure D-79: Impact load applied at 1.9 ft away instead of mid-height of assumed lateral restraint points at top and bottom of the 3rd floor column

Table D-8: Results from loading conditions of Waikīkī residential building exterior shear wall

Moment K-ft	Axial Load Kips	Shear @ d Kips	Load Combination
Floor 1			
196	92.91	84	1.2D+Ftsu+0.5L (Hydro)
196	6.89	84	0.9D+Ftsu (Hydro)
176	92.91	102	1.2D+Ftsu+0.5L (Impact)
176	6.89	102	0.9D+Ftsu (Impact)
Floor 2			
116	79.63	41	1.2D+Ftsu+0.5L (Hydro)
116	5.90	41	0.9D+Ftsu (Hydro)
168	79.63	101	1.2D+Ftsu+0.5L (Impact)
168	5.90	101	0.9D+Ftsu (Impact)
Floor 3			
5	66.36	1	1.2D+Ftsu+0.5L (Hydro)
5	4.92	1	0.9D+Ftsu (Hydro)
116	66.36	100	1.2D+Ftsu+0.5L (Impact)
116	4.92	100	0.9D+Ftsu (Impact)
Floor 4			
1	53.09	0	1.2D+Ftsu+0.5L (Hydro)
1	3.93	0	0.9D+Ftsu (Hydro)
16	53.09	6	1.2D+Ftsu+0.5L (Impact)
16	3.93	6	0.9D+Ftsu (Impact)
Floor 5			
0	39.82	0	1.2D+Ftsu+0.5L (Hydro)
0	2.95	0	0.9D+Ftsu (Hydro)
40	39.82	1	1.2D+Ftsu+0.5L (Impact)
40	2.95	1	0.9D+Ftsu (Impact)
Floor 6			
0	26.54	0	1.2D+Ftsu+0.5L (Hydro)
0	1.97	0	0.9D+Ftsu (Hydro)
10	26.54	0	1.2D+Ftsu+0.5L (Impact)
10	1.97	0	0.9D+Ftsu (Impact)
Floor 7			
0	13.27	0	1.2D+Ftsu+0.5L (Hydro)
0	0.98	0	0.9D+Ftsu (Hydro)
2	13.27	0	1.2D+Ftsu+0.5L (Impact)
2	0.98	0	0.9D+Ftsu (Impact)

D.15.3.2 Existing Exterior Shear Wall Design for Combined Gravity and Tsunami Loads

The shear wall chosen at Grid Line D from **Figure A-16** will now be checked at all levels for combined flexure and axial loads due to gravity and tsunami for load combinations defined in **Section 6.8.3.3**.

Figure D-80 shows the interaction diagram for the typical exterior shear wall including the tsunami load combinations.

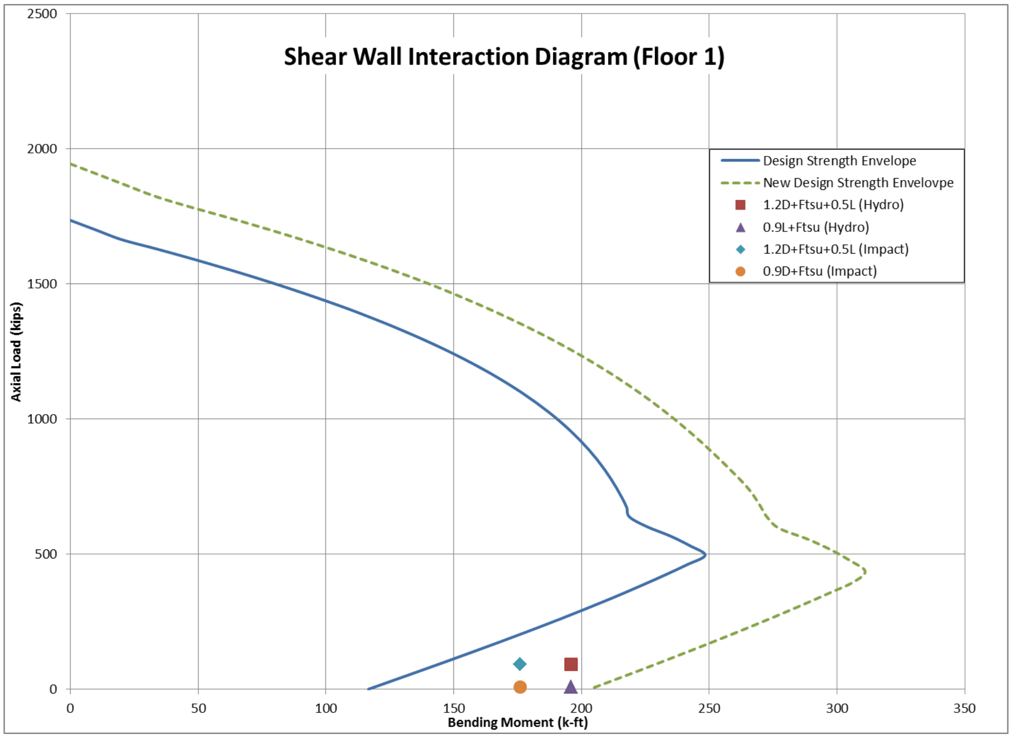


Figure D-80: Interaction diagram for typical ground floor exterior wall segment showing tsunami load combinations

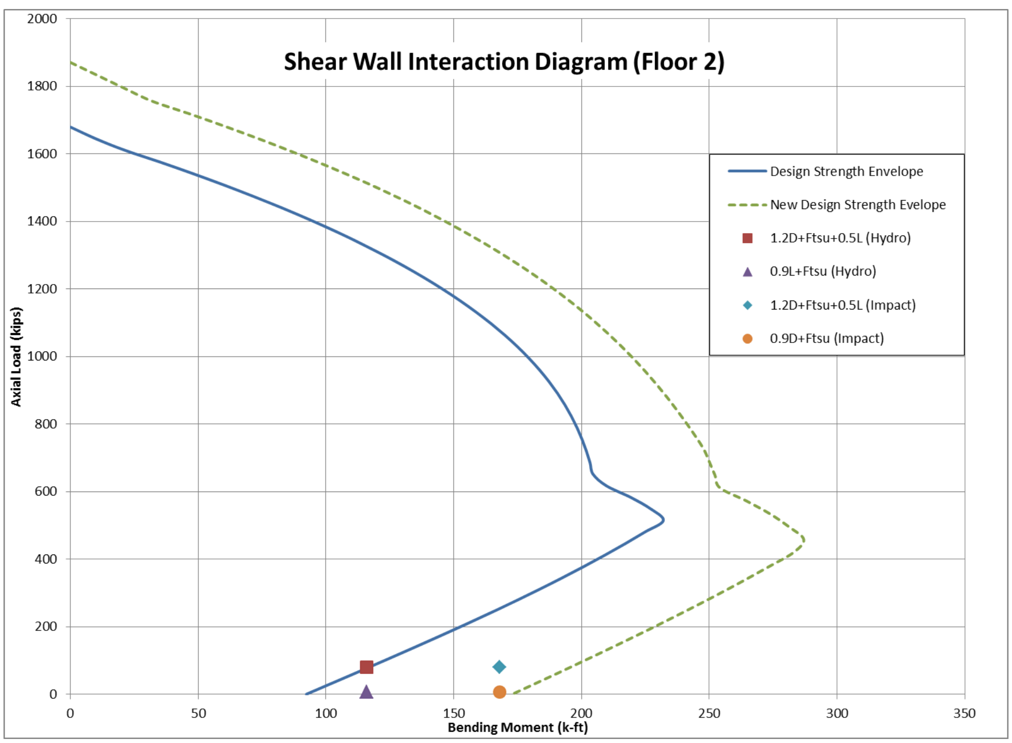


Figure D-81: Interaction diagram for typical 2nd floor exterior wall segment showing tsunami load combinations

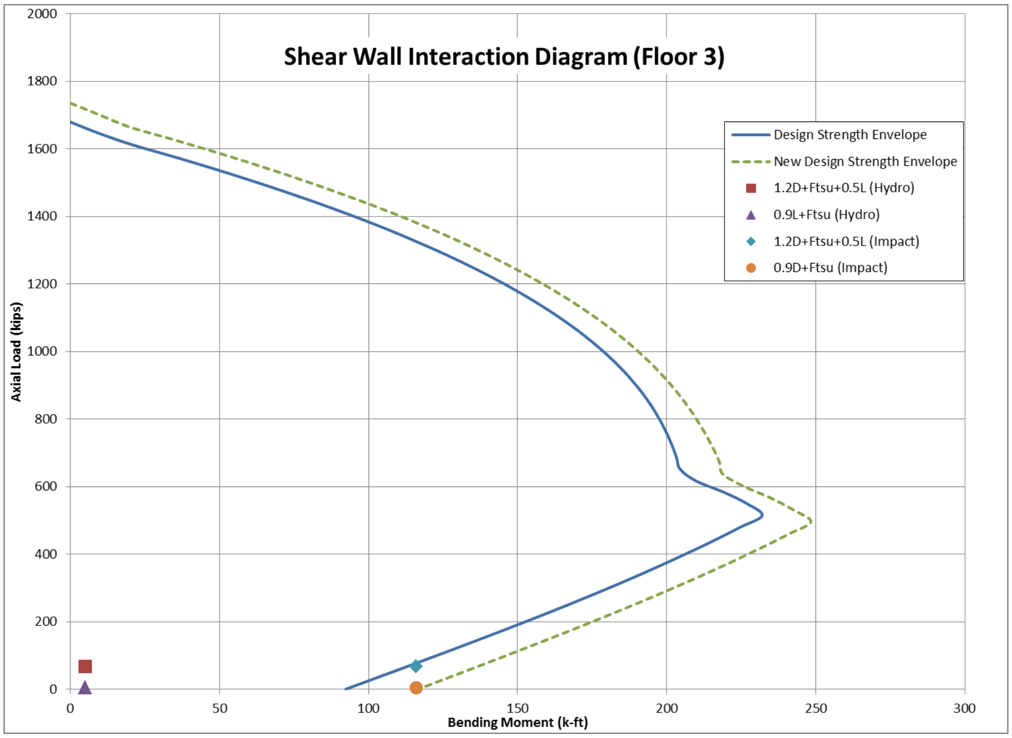


Figure D-82: Interaction diagram for typical 3rd floor exterior wall segment showing tsunami load combinations

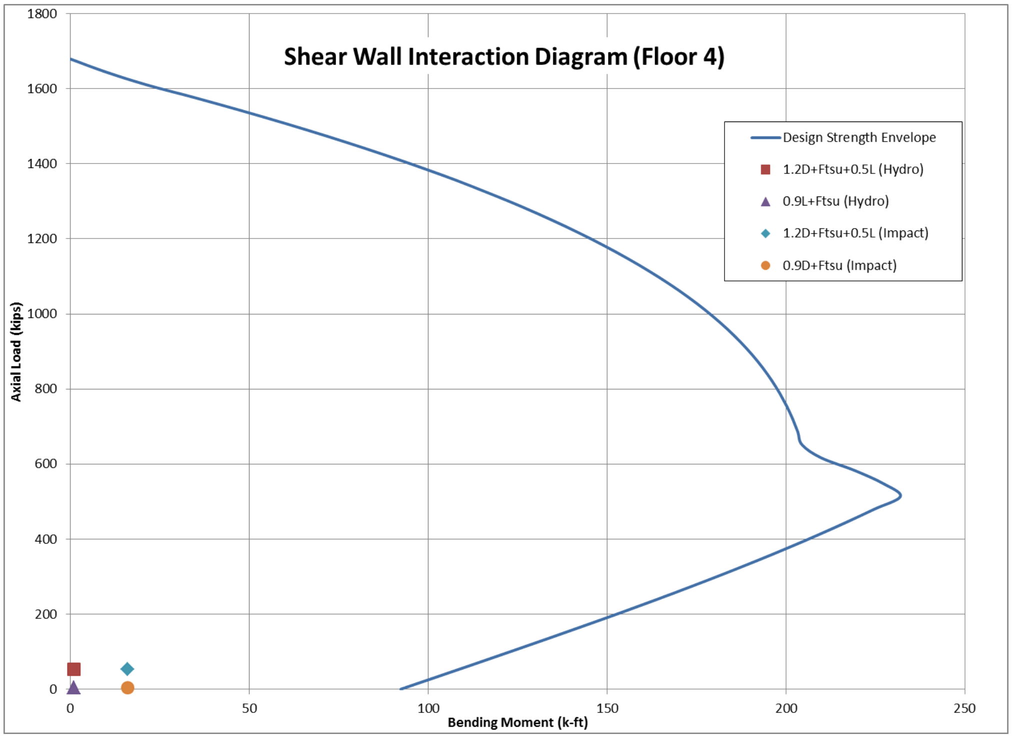


Figure D-83: Interaction diagram for typical 4h floor exterior wall segment showing tsunami load combinations

By inspection the remaining shear walls are adequate to resist the tsunami bending moments.

D.15.3.3 New Typical Shear Wall Design

The interaction diagrams show that all the walls are adequate for bending moments due to hydrodynamic load and derbies impact. Although the floors are adequate for the bending moments Floors 1 – 3 are not adequate for shear loading, **Figure D-84** to **Figure D-86** show the revised wall designs required to resist the tsunami loads. **Figure D-87** shows the side view of the wall with shear stud rails included.

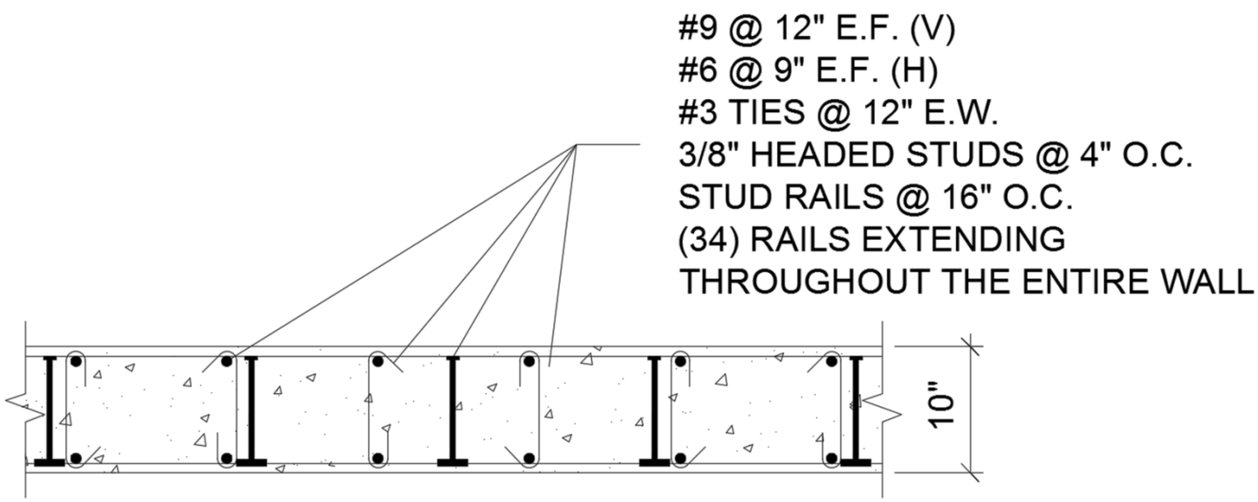


Figure D-84: New exterior wall, cross-section at the ground floor level based on tsunami design requirements.

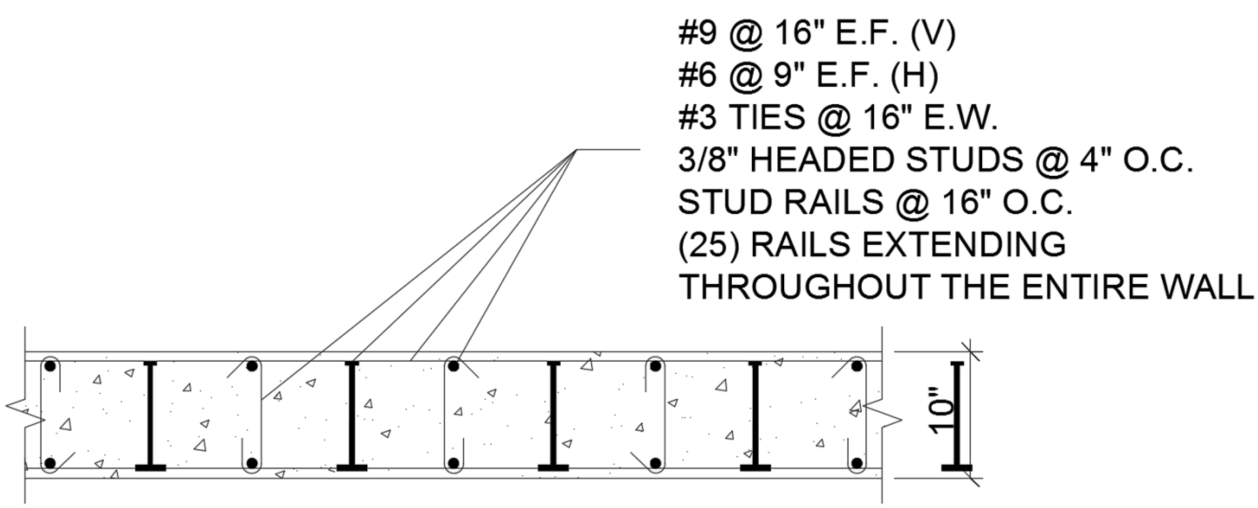


Figure D-85: New exterior wall, cross-section at the 2nd floor level based on tsunami design requirements.

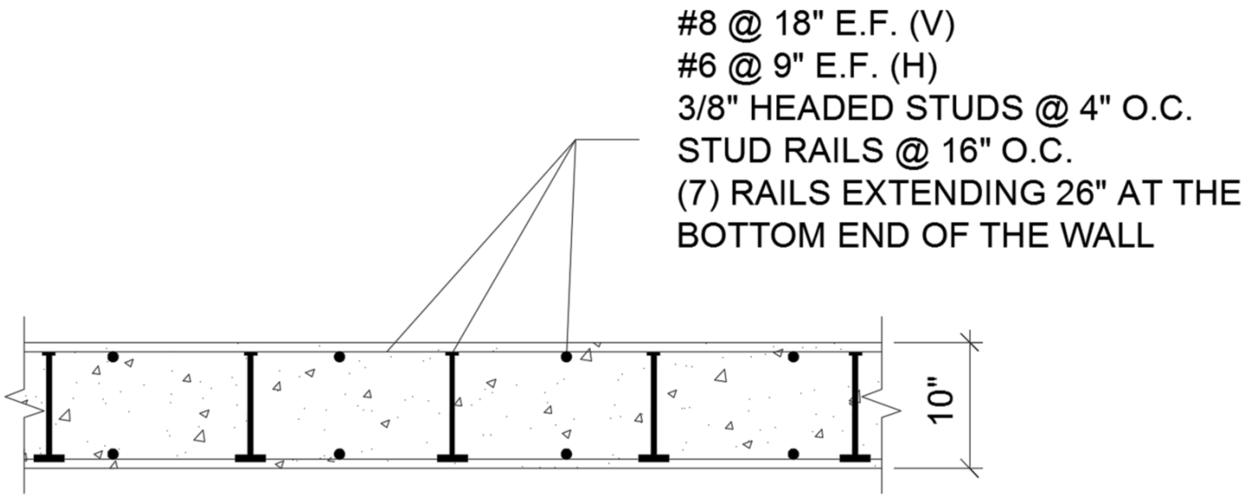


Figure D-86: New exterior wall, cross-section at the 3rd floor level based on tsunami design requirements.

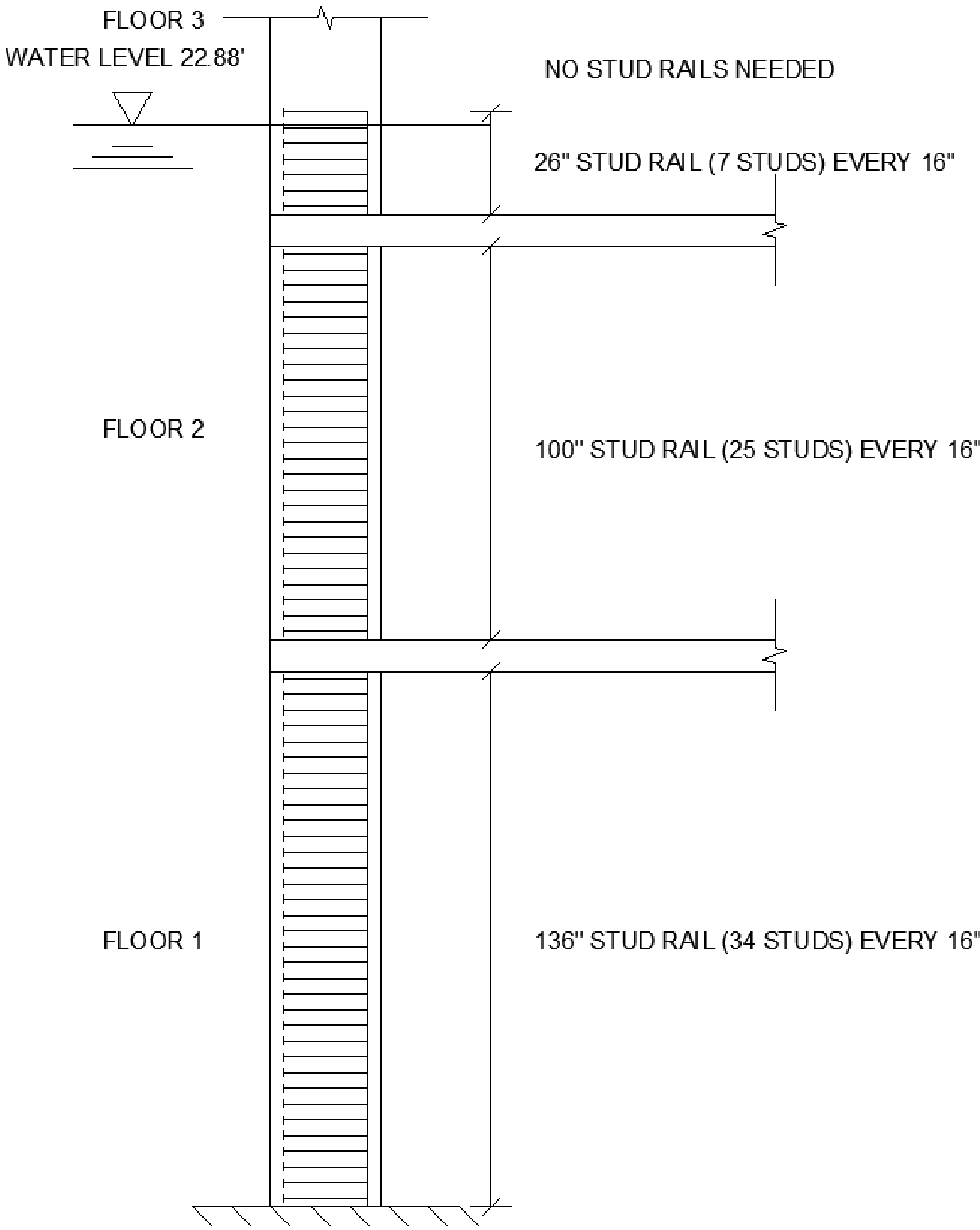


Figure D-87: Stud Rail Diagram for the Floor 1 – 3

D.15.3.4 Exterior Shear Wall Shear Design

Critical Shears:

1^{st} Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$), P_u = 6.89 k:

Shear Capacity of existing shear wall (10" thick):

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\, b\, d = 2\left(1 + \frac{807}{2{,}000\times 68\times 10}\right)\, 1\,\sqrt{4000}\, \times 68 \times 8.75/1{,}000$ = 75 kips

$N_u = (\frac{P_u\times 1{,}000}{I_w}) \times b = \left(\frac{6.89\times 1{,}000}{580}\right) \times 68 = 807\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$ = 0.75 (75+0) = 58 kips

V_{tsu} = 102 kips > $\text{Ø}V_n$ = 56 kips ∴ 61 kips needed

Shear Capacity for New Shear Wall (10" thick):

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\, b\, d = 2\left(1 + \frac{807}{2{,}000\times 68\times 10}\right)\, 1\,\sqrt{4000}\, \times 68 \times 8.75/1{,}000$ = 75 kips

$N_u = (\frac{P_u\times 1{,}000}{I_w}) \times b = \left(\frac{6.89\times 1{,}000}{580}\right) \times 68 = 807\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$\text{Ø}V_n = \text{Ø}(V_C + V_S)$ = 0.75 (75+0) = 56 kips

V_{tsu} = 102 kips > $\text{Ø}V_n$ = 56 kips ∴ 61 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.75}{4}$ = 62 Kips

n = $\frac{Length}{Rail\ Spacing} = \frac{68"}{16"}$ = 4.25

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 = 8.75/2 = 4.375 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_{S\,(needed)}} = \frac{4.25 \times 0.11 \times 60 \times 8.75}{59} = 4.1$ in

$\therefore s_{used} = 4$ in

$ØV_n = Ø(V_C + V_S) = 0.75\ (75 + 62) = 103$ Kips

$ØV_n = 103$ Kips > $V_{Tsu} = 102$ Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ mid\text{-}height} = 63$ Kips > $ØV_c = 54$ Therefor the rails go up the entire wall of the Shear Wall

2nd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** ($0.9D + F_{TSU}$), $P_u = 5.9$ k:

Shear Capacity of existing shear wall (10” thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{692}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.8125/1{,}000 = 76$ kips

$N_u = \left(\frac{P_u \times 1{,}000}{l_w}\right) \times b = \left(\frac{5.9 \times 1{,}000}{580}\right) \times 68 = 692\ lb$

l_w = 28’+11’+11’- 2 x 10” (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76+0) = 57$ kips

$V_{tsu} = 101$ kips > $ØV_n = 57$ kips $\therefore$ 59 kips needed

Shear Capacity for New Shear Wall (10” thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\,b\,d = 2\left(1 + \frac{692}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.8125/1{,}000 = 76$ kips

$N_u = \left(\frac{P_u \times 1{,}000}{l_w}\right) \times b = \left(\frac{5.9 \times 1{,}000}{580}\right) \times 68 = 692\ lb$

l_w = 28’+11’+11’- 2 x 10” (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76+0) = 57$ kips

$V_{tsu} = 101$ kips > $ØV_n = 57$ kips $\therefore$ 59 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.8125}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"}$ = 4.25

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 = 8.8125/2 = 4.41 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{4.25 \times 0.11 \times 60 \times 8.8125}{59}$ = 4.2 in

∴ s_{used} = 4 in

$ØV_n = Ø(V_C + V_S)$ = 0.75 (76 + 62) = 103 Kips

$ØV_n$ = 103 Kips > V_{Tsu} = 101 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ mid\text{-}height}$ = 54 Kips < $ØV_c$ = 57 The rails go up the entire wall of the Shear Wall

3rd Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 4.92 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\ b\ d = 2\left(1 + \frac{577}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{4.92 \times 1{,}000}{580}\right) \times 68 = 577\ lb$

I_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_c + V_S)$ = 0.75 (76+0) = 57 kips

V_{tsu} = 100 kips > $ØV_n$ = 57 kips ∴ 57 kips needed

Shear Capacity for New Shear Wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\ b\ d = 2\left(1 + \frac{577}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000$ = 76 kips

$N_u = (\frac{P_u \times 1{,}000}{I_w}) \times b = \left(\frac{4.92 \times 1{,}000}{580}\right) \times 68 = 577\ lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76 + 0) = 57$ kips

V_{tsu} = 100 kips > $ØV_n$ = 57 kips ∴ 57 kips needed

Shear Stud Reinforcement:

$V_S = \frac{n \times A_v \times f_y \times d}{s} = \frac{4.25 \times 0.11 \times 60 \times 8.875}{4}$ = 62 Kips

$n = \frac{Length}{Rail\ Spacing} = \frac{68"}{16"} = 4.25$

A_v = 3/8" stud = 0.11 in^2

s_{max} = d/2 =8.875/2 = 4.44 in

$s_{needed} = \frac{n \times A_v \times f_y \times d}{V_S} = \frac{3.78 \times 0.11 \times 60 \times 12.875}{57}$ = 4.4 in

∴ s_{used} = 4 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76 + 62) = 104$ Kips

$ØV_n$ = 104 Kips > V_{Tsu} = 100 Kips Therefor the Shear Wall is now adequate for Shear loading

$V_{Tsu\ @\ 10"}$ = 23 Kips < $ØV_c$ = 57 Therefor rails go up 10" (3 Studs) at the bottom end of the Shear Wall

4th Floor:

At the critical axial load per **Eqn. 6.8.3.3.-1b** (0.9*D* + F_{TSU}), P_u = 3.93 k:

Shear Capacity of existing shear wall (10" thick):

$ØV_n = Ø(V_C + V_S)$

Where $V_C = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}\, b\, d = 2\left(1 + \frac{461}{2{,}000 \times 68 \times 10}\right)\ 1\sqrt{4000}\ \times 68 \times 8.875/1{,}000 = 76$ kips

$N_u = (\frac{P_u \times 1{,}000}{l_w}) \times b = \left(\frac{3.93 \times 1{,}000}{560}\right) \times 68 = 461\ lb$

l_w = 28'+11'+11'- 2 x 10" (thickness) = 580 in

$ØV_n = Ø(V_C + V_S) = 0.75\ (76+0) = 57$ kips

V_{tsu} = 1 kips < $ØV_n$ = 57 kips ∴ no shear studs are needed

By inspection the remaining shear walls are adequate to resist the tsunami shear force.

APPENDIX E – ADDITIONAL COST ESTIMATION

This appendix provides an example of how the additional material quantities were computed for the Office and Residential Buildings at the Seaside location.

Office Building

<u>Exterior Columns:</u>

The exterior columns remain the same size (28” x 28”), so there is no increase in concrete quantities, however the reinforcing steel increased for the ground and second floor columns on the coastal and inland sides of the building (total of 20 columns). Because of the ASCE 7 requirement to consider flow directions at 22.5 degrees either side of the normal to the shoreline, the exterior columns on the ends of the building will also be exposed to debris impact loading. Similar strengthening would be required for these columns (total of 12 columns). The increased reinforcement quantities for a single column are therefore multiplied by 32 to account for all of the exterior moment resisting frame columns.

Longitudinal steel:

Floor 1 (14’ tall) - Original column

8#10 rebar: (8*1.27 in^2*168”) = 1,707 in^3 * 0.284 = 484.7 lb x 32 columns = 15,513 lb

Floor 1 (14’ tall) - New column

20#11 rebar: (20*1.56 in^2*168”) = 5,241.6 in^3*0.284 = 1,488.6 lb x 32 columns= 47,636lb

Increase in Longitudinal Steel = 47,636 – 15,513 = **<u>32,123 lb</u>**

Floor 2 (12’ tall) - Original column

8#9 rebar: (8*1 in^2*144”) = 1,152 in^3 * 0.284 = 327.2 lb x 32 columns = 10,469 lb

Floor 2 (12’ tall) - New column

12#11 rebar: (12*1.56 in^2*144”) = 2,695.7 in^3 * 0.284 = 765.6 lb x 32 columns = 24,498

Increase in Longitudinal Steel = 24,498 – 10,469 = **<u>14,029 lb</u>**

Transverse steel:

End Section = [(2*(h/s)*(2(b-2*clear cover)+(2(h-2*clear cover)+hooks)* bar Area] + [number of ties*(b-2*clear cover)* Area of tie]

Center Section = [((L2-2h)/s)*(2(b-2*clear cover)+(2(h-2*clear cover)+hooks)* bar Area] + [number of ties*(b-2* clear cover + hooks)* Area of tie]

Floor 1 (14’ tall) - Original column

End: 3#4 rebar at 4 in O.C.

[(2(28”/4”)*(2(28”-4”)+2(28”-4”)+2*5”)*0.2in^2]+[(2*(28”/4”)*((28”-4”)+(28”-4”)+5”*4)*0.2 in^2] =296.8 in^3+ 190.4 in^3 = 487.2 in^3 * 0.284 lb/in^3 = 138.4 lb x 32 columns = 4,428 lb

Center: 3#3 rebar at 6 in O.C.

[((168”-2*28”)/6”)*(2(28”-4”)+2(28”-4”)+2*5”)*0.11 in^2]+[(168-2*28”)/6”)*((28”-4”)+(28”-4”)+5”*4)*0.11 in^2] = 217.7 in^3+ 139.6 in^3 = 357.3 in^3 * 0.284 lb/in^3 = 101.5 lb x 32 columns = 3,247 lb

Total Transverse Steel = 4,428 + 3,247 = 7,675 lb

Floor 1 (14’ tall) - New Column

End: 4#5 rebar at 4 in O.C.

[(2(28”/4”)*(2(28”-4”)+2(28”-4”)+2*5”)*0.31 in^2]+[(2*(28”/4”)*((2*(28”-4”)+2*(28”-4”)+5”*8)*0.31 in^2] = 460 in^3+ 590.2 in^3 = 1050.2 in^3 * 0.284 lb/in^3 = 298.3 lb x 32 columns = 9,545 lb

Center: 4#4 rebar at 6 in O.C.

[((168”-2*28”)/6”)*(2(28”-4”)+2(28”-4”)+2*5”)*0.2 in^2]+[(168-2*28”)/6”)*(2*(28”-4”)+2*(28”-4”)+5”*8)*0.2 in^2] = 395.7 in^3+ 507.7 in^3 = 903.4 in^3 * 0.284 lb/in^3 = 526.6 lb x 32 columns = 8,210 lb

Total Transverse Steel = 9,545 + 8,210= 17,755 lb

Increase in Transverse Steel = 17,755 – 7,675 = ***<u>10,080 lb</u>***

Floor 2 (12’ tall) - Original column

End: 3#4 rebar at 4 in O.C.

[(2(28”/4”)*(2(28”-4”)+2(28”-4”)+2*5”)*0.2in^2]+[(2*(28”/4”)*(28”-4”)+(28”-4”)+5”*4)*0.2 in^2] = 296.8 in^3+ 190.4 in^3 = 487.2 in^3 * 0.284 lb/in^3 = 138.4 lb x 32 columns = 4,428 lb

Center: 3#3 rebar at 6 in O.C.

[((144”-2*28”)/6”)*(2(20”-4”)+2(20”-4”)+2*5”)*0.11 in^2]+[(108-2*20”)/5”)*((20”-4”)+(20”-4”)+5”*4)*0.11 in^2] = 171 in^3+ 109.7 in^3 = 280.7 in^3 * 0.284 lb/in^3 = 79.7 lb x 32 columns = 2,551 lb

Increase in Transverse Steel = 4,428 + 2,551 = 6,979 lb

Floor 2 (12' tall) - New column

End: 4#5 rebar at 4 in O.C.

[(2(28"/4")*(2(28"-4")+2(28"-4")+2*5")*0.31 in^2]+[(2*(28"/4"))*(2*(28"-4")+2*(28"-4")+5"*8)*0.31 in^2] = 460 in^3+ 590.2 in^3 = 1050.2 in^3 * 0.284 lb/in^3 = 298 lb x 32 columns = 9,545 lb

Center: 4#4 rebar at 6 in O.C.

[((144"-2*28")/6")*(2(28"-4")+2(28"-4")+2*5")*0.2 in^2]+[(144-2*28")/6 ")*((28"-4")+(28"-4")+5"*8)*0.2 in^2] = 310.9 in^3+ 398.9.8 in^3 = 709.8 in^3 * 0.284 lb/in^3 = 201.6 lb x 32 columns = 6,451 lb

Total Transverse Steel = 9,545 + 6,451 = 15,996 lb

Increase in Transverse Steel = 15,996 – 6,979 = <u>***9,017 lb***</u>

<u>Interior Columns:</u>

No change in materials.

<u>Combined total increase</u>

Concrete: No increase

Steel:

Total added longitudinal steel: 32,123 + 14,029 = 46,152 lb

Total added transverse steel: 10,080 + 9,017 = 19,097 lb

Total added steel: 46,152 + 19,097 = 65,249 lb

Cost of material, fabrication and installation: 56,249 lbs x 1.5 $/lb = **$ 97,873.50**

<u>Total Office Building cost</u>

Original Structural cost:	$ 7,563,737.50
New Structural cost:	$ 7,661,611.00
Structural % increase:	<u>1.29 %</u>
Original Overall Building cost:	$ 25,212,458.50
New Overall cost:	$ 25,310,332.00
Overall % increase:	<u>***0.39%***</u>

Residential Building

<u>Exterior Columns:</u>

The exterior gravity load columns increase in size from 20" x 20" to 26" x 26" to resist the tsunami loads. The reinforcing steel also increased for the exterior columns at the first three floors columns (total of 16 columns).

Concrete quantity:

Floor 1 (12' tall) - Original column

20"x20" = (20"*20"*144")/ 46,656 in^3/yd^3 = 1.24 yd^3 x 16 columns = 19.8 yd^3

Floor 1 (12' tall) - New column

26"x26" = (26"*26"*144")/ 46,656 in^3/yd^3 = 2.09 yd^3 x 16 columns = 33.4 yd^3

Increase in concrete volume = 33.4 – 19.8 = <u>***13.6 yd³***</u>

Floor 2 (9' tall) - Original column

20"x20" = (20"*20"*108")/ 46,656 in^3/yd^3 = 0.93 yd^3 x 16 columns = 14.9 yd^3

Floor 2 (9' tall) - New column

26"x26" = (26"*26"*108")/ 46,656 in^3/yd^3 = 1.56 yd^3 x 16 columns = 25 yd^3

Increase in concrete volume = 25 – 14.9 = <u>***10.1 yd^3***</u>

Longitudinal steel:

Floor 1 (12' tall) - Original column

8#7 rebar: (8*0.6 in^2*144") = 691 in^3 * 0.284 lb/in^3 = 196 lb x 16 columns = 3,136 lb

Floor 1 (12' tall) - New column

8#11 rebar: (8*1.56 in^2*144") = 1,797 in^3 * 0.284 lb/in^3 = 510 lb x 16 columns = 8,160 lb

Increase in Longitudinal Steel = 8,160 – 3,136 = <u>***5,024 lb***</u>

Floor 2 (9' tall) - Original column

8#7 rebar: (8*0.6 in^2*108") = 518in^3 * 0.284 lb/in^3 = 147.112 lb x 16 columns = 2,354 lb

Floor 2 (9' tall) - New column

8#9 rebar: (8*1 in^2*108") = 864 in^3 * 0.284 lb/in^3 = 245.376 lb x 16 columns = 3,926 lb

Increase in Longitudinal Steel = 3,926 – 2,354 = ***1,572 lb***

Transverse steel:

End = [(2*(h/s)*(2(b-2*clear cover)+(2(h-2*clear cover)+hooks)* bar Area] + [number of ties*(b-2*clear cover)* Area of tie]

Center = [((L2-2h)/s)*(2(b-2*clear cover)+(2(h-2*clear cover)+hooks)* bar Area] + [number of ties*(b-2* clear cover + hooks)* Area of tie]

Floor 1 (12' tall) - Original column

End: 3#4 rebar at 4 in O.C.

[(2(20"/4")*(2(20"-4")+2(20"-4")+2*5")*0.2in^2]+[(2*(20"/4")*(1*(20"-4")+(20"-4")+5"*4)*0.2 in^2] = 148 in^3+ 104 in^3 = 252 in^3 * 0.284 lb/in^3 = 71.6 lb x 16 columns = 1,146 lb

Center: 3#3 rebar at 5 in O.C.

[((144"-2*20")/5")*(2(20"-4")+2(20"-4")+2*5")*0.11 in^2]+[(144-2*20")/5")*((20"-4")+(20"-4")+5"*4)*0.11 in^2] = 169.3 in^3+ 119 in^3 = 288.3 in^3 * 0.284 lb/in^3 = 81.9 lb x 16 columns = 1,310 lb

Total Transverse Steel = 1,146 + 1,310 = 2,456 lb

Floor 1 (12' tall) - New column

End: 4#4 rebar at 4 in O.C.

[(2(26"/4")*(2(26"-4")+2(26"-4")+2*5")*0.2in^2]+[(2*(26"/4")*(2*(26"-4")+2*(26"-4")+5"*8*0.2 in^2] = 254.8 in^3+ 332.8 in^3 = 587.6 in^3 * 0.284 lb/in^3 = 166.9 lb x 16 columns = 2,670 lb

Center: 3#4 rebar at 5 in O.C.

[((144"-2*26")/5")*(2(26"-4")+2(26"-4")+2*5")*0.2 in^2]+[(144-2*26")/5")*((26"-4")+(26"-4")+5"*4)*0.2 in^2] = 360.6 in^3+ 235.5 in^3 = 596.2 in^3 * 0.284 lb/in^3 = 169.3 lb x 16 columns = 2,709 lb

Total Transverse Steel = 2,670 + 2,709 = 5,379 lb

Increase in Transverse Steel = 5,379 – 2,456 = ***2,923 lb***

Floor 2 (9' tall) - Original column

End: 3#4 rebar at 4 in O.C.

[(2(20”/4”)*(2(20”-4”)+2(20”-4”)+2*5”)*0.2in^2]+[(2*(20”/4”)*(1*(20”-4”)+(20”-4”)+5”*4)*0.2 in^2] = 148 in^3+ 104 in^3 = 252 in^3 * 0.284 lb/in^3 = 71.6 lb x 16 columns = 1,146 lb

Center: 3#3 rebar at 5 in O.C.

[((108”-2*20”)/5”)*(2(20”-4”)+2(20”-4”)+2*5”)*0.11 in^2]+[(108-2*20”)/5”)*((20”-4”)+(20”-4”)+5”*4)*0.11 in^2] = 110.7 in^3+ 77.8 in^3 = 188.5 in^3 * 0.284 lb/in^3 = 53.5 lb x 16 columns = 856 lb

Total Transverse Steel = 1,146 + 856 = 2,002 lb

Floor 2 (9’ tall) - New column

End: 3#4 rebar at 4 in O.C.

[(2(26”/4”)*(2(26”-4”)+2(26”-4”)+2*5”)*0.2in^2]+[(2*(26”/4”))*((26”-4”)+(26”-4”)+5”*4)*0.2 in^2] = 254.8 in^3+ 166.4 in^3 = 421.2 in^3 * 0.284 lb/in^3 = 119.6 lb x 16 columns = 1,914 lb

Center: 3#3 rebar at 5 in O.C.

[((108”-2*26”)/5”)*(2(26”-4”)+2(26”-4”)+2*5”)*0.2 in^2]+[(108-2*26”)/5”)*((26”-4”)+(26”-4”)+5”*4)*0.2 in^2] = 120.7 in^3+ 78.8 in^3 = 199.5 in^3 * 0.284 lb/in^3 = 56.7 lb x 16 columns = 907 lb

Total Transverse Steel = 1,914 + 907 = 2,821 lb

Increase in Transverse Steel = 2,821 – 2,002 = ***819 lb***

Interior Column:

No change in materials

Walls:

Concrete: No change

Longitudinal steel:

Floor 1 (12’ tall, 28’ wide) - Original wall

#8 @ 18” rebar: (144*(2*(336”/18”)) *0.79 in^2) = 4,247 in^3 * 0.284 lb/in^3 = 1,206 lb x 2 walls = 2,412 lb

Floor 1 (12’ tall, 28’ wide) - New wall

#10 @ 12” rebar: (144*(2*(336”/12”)) *1.27 in^2) = 10,241 in^3 * 0.284 lb/in^3 = 2,909 lb x 2 walls = 5,817 lb

Increase in Longitudinal Steel = 5,817 – 2,412 = **3,405 lb**

Floor 2 (9’ tall, 28’ wide) - Original column

#7 @ 18” rebar: (108*(2*(336”/18”)) *0.6 in^2) = 2,419.2 in^3 * 0.284 lb/in^3 = 687 lb x 2 walls = 1,374 lb

Floor 2 (9’ tall, 28’ wide) - New column

#9 @ 12” rebar: (108*(2*(336”/12”)) *1 in^2) = 6,048 in^3 * 0.284 lb/in^3 = 1,718 lb x 2 walls = 3,435 lb

Increase in Longitudinal Steel = 3,435 - 1,374 = **2,061 lb**

Floor 3 (9’ tall, 28’ wide) - Original column

#6 @ 18” rebar: (108*(2*(336”/18”)) *0.44 in^2) = 1,774 in^3 * 0.284 lb/in^3 = 503.8 lb x 2 walls = 1,008 lb

Floor 3 (9’ tall, 28’ wide) - New column

#9 @ 12” rebar: (108*(2*(336”/12”)) *1 in^2) = 6,048 in^3 * 0.284 lb/in^3 = 1,718 lb x 2 walls = 3,435 lb

Increase in Longitudinal Steel = 3,435 – 1,008 = **2,427 lb**

Transverse steel:

Floor 1 (12’ tall, 28’ wide) - Original wall

No Shear studs required

Floor 1 (12’ tall, 28’ wide) - New wall

#of Studs x # of Rails x 2

33 x 21 x 2 = 1,386 studs x 2 lb/stud = 2,772 lb x 2 walls = **5,544 lb**

Floor 2 (9’ tall, 28’ wide) - Original wall

No Shear studs required

Floor 2 (9’ tall, 28’ wide) - New wall

24 x 21 x 2 = 1,008 studs x 2 lb/studs = 2,016 lb x 2 walls = **4,032 lb**

Floor 3 (9' tall, 28' wide) - Original wall

No Shear studs required

Floor 3 (9' tall, 28' wide) - New wall

24 x 21 x 2 = 1,008 studs x 2 lb/studs = 2,016 lb x 2 walls = **4,032 lb**

Floor 3 (9' tall, 28' wide) - Original wall

No Shear studs required

Floor 3 (9' tall, 28' wide) - New wall

8 x 21 x 2 = 336 studs x 2 lb/studs = 672 lb x 2 walls = **1,344 lb**

<u>Combined total increase</u>

Concrete: $13.6\ yd^3 + 10.1\ yd^3 = 23.7\ yd^3$

$23.7\ yd^3 \times 1{,}095\ \$/yd^3 =$ **$ 25,952.00**

Steel:

Exterior Column:

Total added Longitudinal steel: 5,024 + 1,572 = 6,596 lb

Total added Transverse steel: 2,923 + 819 = 3,742 lb

Total added steel: 6,596 + 3,742 = 10,338 lb

Shear Walls:

Total added Longitudinal steel: 3,405 + 2,061 + 2,427 = 7,893 lb

Total added Transverse steel: 5,544 + 4,032 + 4,032 + 1,344 = 14,942 lb

Total added steel: 7,893 + 14,942 = 22,835 lb

Overall Steel:

Total added steel: 10,338 + 22,835 lb = 33,173 lb

33,173 lbs x 1.5 $/lb = **$ 49,760.00**

Total = $ 25,952.00 + $ 49,760.00 = $ 75,712

Total Residential Building cost

Original Structural cost:	$ 5,826,429.58
New Structural cost:	$ 5,902,324.89
Structural % increase:	1.30 %
Original Overall Building cost:	$ 23,305,718.33
New Overall cost:	$ 23,381,613.64
Overall % increase:	***0.33 %***